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ABSTRACT

This report contains four papers written during an investigation of methods for evaluating the safety of existing school buildings under Research Applied to National Needs (RANN) grants. In "Evaluation of Earthquake Safety of Existing Buildings," by B. Bresler, preliminary ideas on the evaluation of the earthquake safety of existing buildings are described. The second paper, "Assessment of Earthquake Safety and of Hazard Abatement," by B. Bresler, T. Okada, and D. Zisling, discusses methods for assessing the seismic safety of structures and procedures for establishing priorities for evaluating and abating hazards are indicated. The third paper, "Seismic Safety of Existing Low-Rise Reinforced Concrete Buildings," by T. Okada and B. Bresler, describes a methodology for evaluating the seismic safety of low-rise reinforced concrete buildings and its application to existing school buildings. The fourth paper, "Design and Engineering Decisions: Failure Criteria (Limit States)," by V. Bertero and B. Bresler, discusses the failure criteria (inadmissable limit states) that should be considered in aseismic design of buildings. (Author/MLF)

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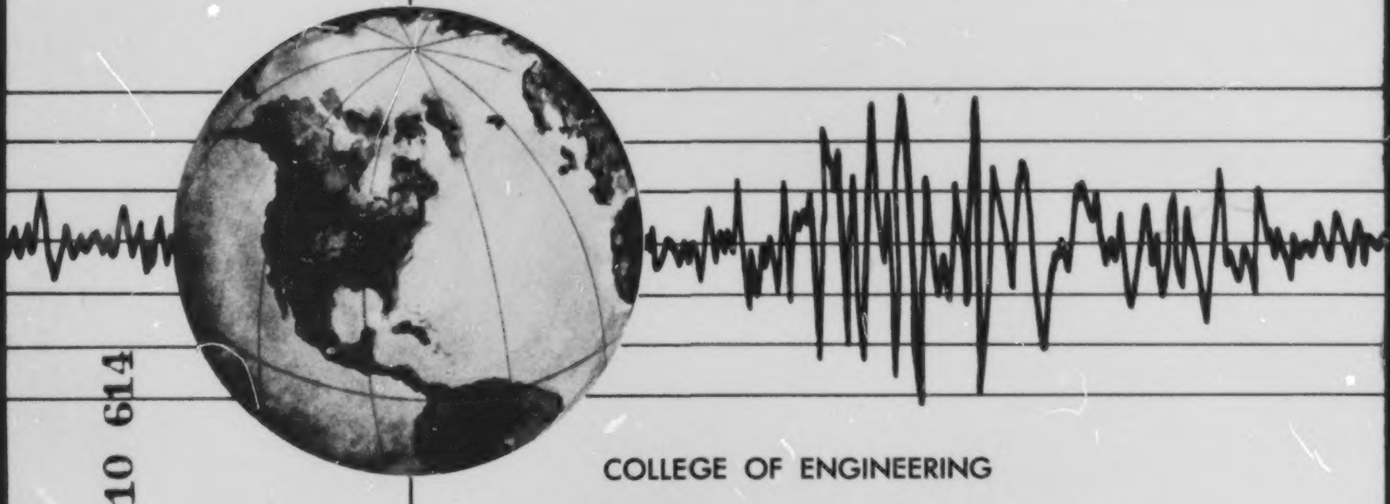
DEVELOPING METHODOLOGIES FOR EVALUATING THE EARTHQUAKE SAFETY OF EXISTING BUILDINGS

Report to the National Science Foundation



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Evaluation of Earthquake Safety of Existing Buildings
B. Bresler

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V. V. Bertero & B. Bresler

Report to:

National Science Foundation
Research Applied to National Needs (RANN)

Report No. UCB/EERC-77/06
Earthquake Engineering Research Center
College of Engineering
University of California
Berkeley, California

February 1977

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PREFACE

This report contains four papers written during an investigation of methods for evaluating the safety of existing school buildings under NSF RANN (Research Applied to National Needs) Grants. These papers are not readily available to researchers and engineers in the United States and are therefore issued as a single Earthquake Engineering Research Center report.

The first paper, EVALUATION OF EARTHQUAKE SAFETY OF EXISTING BUILDINGS, by B. Bresler, was presented at the U.S.-Japan Seminar on Earthquake Engineering with Special Emphasis on Reinforced Concrete Structures in Berkeley, California, September 4-8, 1973. The second and third papers, ASSESSMENT OF EARTHQUAKE SAFETY AND OF HAZARD ABATEMENT, by B. Bresler, T. Okada, and D. Zisling, and SEISMIC SAFETY OF EXISTING LOW-RISE REINFORCED CONCRETE BUILDINGS, by T. Okada and B. Bresler, were presented at a Review Meeting of the U.S.-Japan Cooperative Research Program in Earthquake Engineering with Emphasis on the Safety of School Buildings in Honolulu, Hawaii, August 18-20, 1975, and were published in the Proceedings of that meeting. The fourth paper, DESIGN AND ENGINEERING DECISIONS: FAILURE CRITERIA (LIMIT STATES), by V. Bertero and B. Bresler, was presented at the 6th World Conference on Earthquake Engineering in New Delhi, January 10-19, 1977, and will be published in the Proceedings of that Conference.

This collection of papers in a single report should be useful to various investigators interested in methodologies for evaluating the seismic safety of existing buildings.

EVALUATION OF EARTHQUAKE SAFETY OF EXISTING BUILDINGS

by

B. Bresler

Professor of Civil Engineering
Department of Civil Engineering
University of California, Berkeley

Paper presented at the U.S.-Japan Seminar on Earthquake Engineering with Special Emphasis on Reinforced Concrete Structures, Berkeley, California, September 4-8, 1973.

EVALUATION OF EARTHQUAKE SAFETY OF EXISTING BUILDINGS

Boris Bresler[1]

1. INTRODUCTION

In a recent NBS publication [1] on building practices for disaster mitigation, the following statements were used to support recommendations concerning safety evaluations of existing buildings:

> In addition to insuring safety of new construction, it is important to ascertain the safety of the large inventory of existing buildings . . .
>
> There presently are no nationally recognized effective systematic and economical procedures for assessing the hazards of existing buildings for future extreme loads . . . Appropriate evaluation methods should be developed for systematic predisaster surveys of safety for long term use . . . Criteria for acceptance or abatement of hazards should be capable of reflecting the responsible authorities assessment of the social and economic consequences of action . . .

In the present paper, preliminary ideas on the evaluation of the earthquake safety of existing buildings are described. The principal differences between designing for safety and evaluating safety are distinguished, and recent programs for evaluating the safety of existing buildings are briefly summarized. Finally, a possible approach for developing general guidelines for evaluating safety is proposed.

2. EARTHQUAKE RESISTANT DESIGN AND EVALUATION OF EARTHQUAKE SAFETY

Design is a conceptual process. As long as a building exists only on paper (drawings, calculations, specifications), actual physical properties of building components cannot be determined. While the dynamic characteristics of an existing building can be measured, these properties must be estimated for a building that exists as a design.

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In design, an engineer must account for a number of uncertainties. For example, the quality of materials and workmanship eventually realized in constructing a building is uncertain until that structure is completed, and may be either superior or inferior to that specified. In an existing building, the quality of materials can be estimated more closely from field control records, and corroborated by sampling and laboratory testing.

Design of a building is governed by the then-current state-of-the-art of engineering. As understandings of structural behavior and design criteria change during the life of a building, analysis and judgment regarding the safety of that building may differ from the analysis and judgment recorded by the engineer during the original design process.

Determining safety, on the other hand, involves evaluating the risk of damageability and collapse, and human and economic consequences of possible disasters. Both risk of damageability and relative cost of economic decisions change in time, as for example risk may increase with age and relative economy of repair vs. replacement may change. Thus, the degree of safety assigned to a given building may also change. For example, a new building properly designed for a fifty-year life would rate a high degree of safety when constructed. Fifty years after construction, that same building may or may not be assigned a lower degree of safety.

The following remarks, quoted from the 1967 SEAOC Commentary on Recommended Lateral Force Requirements [2], illustrate the preceding argument:

> The SEAOC Code provides minimum design criteria in specific categories, but in broad general terms. Reliance must be placed on the experienced structural engineer to interpret and adapt the basic principles to each specific structure. Because of the great number of variables and the complexity of the problem it is impractical and beyond the scope of the Code to go to such detail to cover specifically all the variations in response, dynamic characteristics of the structure, variables in ground motion, the intensity of the earthquake, the distance to the epicenter of the seismic disturbance,

> and the type of soil . . . Thus latitude for the exercise of analysis and judgment must be given the responsible structural engineer . . . If the objective of the seismic design of a particular structure is something more than that for which the code is intended, the structural engineer must establish criteria to suit the specific problem . . .

These remarks suggest a basis for differentiating between designing for safety and evaluating safety. While it is quite proper to accept a design that complies with an appropriate code and is recommended by a responsible engineer, these criteria do not suffice as a basis for reliably evaluating the earthquake safety of existing buildings.

3. THE LONG BEACH, CALIFORNIA, STUDY

The hazard posed by older buildings in the event of a strong earthquake is a serious problem in many California communities. In January 1970, the City of Long Beach authorized a special study of this problem by J. H. Wiggins Co. and D. F. Moran, special consultant. A report on their study [3] was published in 1971. The following summary highlights portions of the report without detailing the specific methodology for earthquake risk analysis.

The report describes a rational, balanced risk design concept, and a basically empirical, judgmental procedure for grading existing buildings. The report also proposes a model for an earthquake hazard ordinance for designing new structures and rehabilitating existing structures.

The balanced risk design concept is introduced for designing new structures and for determining the extent of strengthening necessary to render existing structures safe. An empirical ("uniform, systematic, and practical") procedure for ascertaining earthquake hazard is also incorporated into the ordinance. The report emphasizes that quantifiable arguments of risk are at best a mechanism for judgment and compromise. A great number of social, economic, and indeed human factors must be considered in attempting to remedy earthquake hazard. The balanced risk approach suggested in the Long Beach Study is just one factor in reaching a decision in a given situation.

The concept of balanced risk is based on selecting a basic risk rate, building life, an importance factor for a given structure, and then, with due regard for local seismicity, arriving at a design lateral force intensity for a specific building.

Assuming that a relationship between lateral force V and structural weight W can be expressed by a base shear coefficient C_d such that $V = C_dW$, the report proposed a relationship between C_d, the life of a structure L, death risk due to earthquake, the importance factor for a structure I, and local seismicity.

Thus considering a typical one- or two-story building on the lowest risk soil type in Long Beach, the report proposed the following values for C_d:

DESIGN LATERAL FORCE COEFFICIENT C_d

Basic Death Risk Rate (per year)	Building Life (years)	Importance Factor			
		1	2	3	4
		10^{-2}	10^{-1}	1	10
		Basic	Basic	Basic	Basic
1×10^{-7}	5	0.106	0.079	0.066	0.053
	10	0.132	0.106	0.079	0.066
	40	0.251	0.211	0.158	0.132
1×10^{-6}	5	0.079	0.066	0.053	0.040
	10	0.106	0.079	0.066	0.053
	40	0.211	0.158	0.132	0.092
1×10^{-5}	5	0.066	0.053	0.040	0.026
	10	0.079	0.066	0.053	0.040
	40	0.158	0.132	0.092	0.066

An empirical procedure for grading existing buildings was suggested to formalize the professional judgment of engineers experienced in aseismic design and observation of earthquake damage. A scale of

0 to 180 points was proposed for grading buildings. A rating of 0 to 50 points reflected low hazard, and no modification or strengthening was required. A rating of 51 to 100 points indicated intermediate hazard; some modification or strengthening was required. Finally, a rating of 101 to 180 points reflected serious life hazard and major strengthening or demolition was required.

The grading system was based on evaluations of five characteristic items: (1) framing system and/or walls (0-40 points), (2) diaphragm and/or bracing system (0-20 points), (3) partitions (0-20 points), (4) special hazards such as shape, soil condition, poorly anchored components, vulnerable mechanical and electrical services (0-50 points), and (5) physical condition (0-50 points). Detailed suggestions for grading specific items were included in the report, but gradings would of course vary with individual judgment.

4. SAN FRANCISCO SCHOOL BUILDING STUDY

Legislation passed in 1967 required that pre-1933 (Field Act) California schools be inspected and rated as safe or unsafe based on protection of life and prevention of personal injury at a level of safety established by the Field Act. The legislation required that unsafe schools either be rehabilitated or abandoned as soon as possible, but no later than June 30, 1975.

In 1969, the City of San Francisco authorized evaluation of all pre-1933 school buildings. These evaluations [[illegible]] were carried out by various registered structural engineers who were asked to provide the following information:

(1) Whether the school building was legally safe or unsafe as defined in the Education Code, State of California.

(2) Criteria used in arriving at the above conclusion.

(3) Adequacy of vertical load and lateral force-resisting systems.

(4) Results of any field investigations, materials testing, soil and foundation investigations, etc.

(5) Description of recommended rehabilitation.

(6) Detailed cost estimate of rehabilitation.

In addition to a number of buildings in nine junior and senior high schools, fifty-three elementary schools were investigated. Coverage of vital information varied in the engineering reports on these investigations. Many reports assumed that the buildings conformed to recorded information, and recommendations were based on calculations ascertaining compliance of the design with current legal code standards (Title 24). Some indication of the type of schools involved and the economic scope of the problem is provided by the following.

Of fifty-three elementary schools, twenty-three were of reinforced concrete, three had steel frames or were of steel frame - concrete composite construction, and twenty-seven were of wood construction. The average floor area of the reinforced concrete elementary schools was 38,800 sq. ft.; the total cost of rehabilitating this group of schools was estimated to be $5,200,000 (in 1969), or approximately $5.80 per sq. ft. These schools were from fifty-four to thirty-nine years old (in 1969). The lowest rehabilitation cost, $1.63 per sq. ft., was projected for the oldest reinforced concrete school built in 1915.

Clearly, some buildings are stronger and some weaker than the reports indicated. While the engineers' evaluations represented the state-of-the-art, uncertainties with respect to their reports are substantial, and fall into several categories:

(1) Intensity, duration, and other ground motion characteristics at a given site are difficult to establish. Distance from the source of an earthquake and local site conditions -- e.g. geological configuration and physical condition of the soil, building foundation, type of building, and orientation with respect to an earthquake source -- greatly influence earthquake action upon a structure. Seismic coefficients prescribed by current codes do not account for many of these factors.

(2) The actual (hidden) quality of materials and workmanship, hazards or benefits due to performance of nonstructural elements (partitions, cladding, mechanical or electrical services, ornamentation),

time of earthquake and variations in occupancy with time (daily, seasonal), and the possibility of special hazards (glass fragmentation, explosions, release of toxic chemicals, fire) may greatly alter the safety or unsafety of a given school building.

(3) In most cases, investigation of these problems is complicated by difficulties in ascertaining essential information.

5. PROCESS FOR EVALUATING SAFETY OF AN EXISTING BUILDING

The principal features of the process for evaluating the safety of an existing building are outlined below.

(A) Review of documentation related to an existing building.

(B) Field studies, including observation, basic measurements, testing (laboratory and field), and evaluation of field data. Potential risk of concomitant hazards (e.g. fire, release of toxic materials, etc.).

(C) Analytical studies, including dual earthquake criteria, choice of mathematical model and analytical method, and choice of local site earthquake intensity.

(D) Possible methods of reducing the risk of damage and collapse, perhaps including further analytical studies.

5A. Review of Documentation

Much of the data necessary to evaluate the safety of an existing building must come from reviewing documentation related to building design, construction, and subsequent service life.

(1) Complete set of drawings, including architectural, structural, and mechanical and electrical drawings.

(2) A set of project specifications, usually part of the construction contract and containing information on material specifications, workmanship, and quality control requirements.

(3) Design calculations, principally including structural calculations.

(4) Reports of foundation studies, including site borings and soil test data.

(5) Inspection records, reflecting time-history of construction (dates of inspection of various stages of construction, any notes on variance from drawings and specifications). Any change orders approved by architects and engineers.

(6) Materials testing records, often forming a portion of inspection records. These records should include data on field control tests of concrete, mill or laboratory tests of steel reinforcement, and other laboratory tests (e.g. special details such as welding, mechanical splices, etc.). Statistical evaluation of field control data is desirable.

(7) Records of changes and/or modifications after construction, available from city building officials, from architects and engineers, or from owners. Records of maintenance and/or repair may provide data on special conditions during service life of the structure.

(8) Service history data, including loading history, prior overloads (excessive gravity or wind loads; earthquakes; fire; extreme environmental conditions including temperature, humidity, chemically aggressive conditions; accidents; etc.).

5B. Field Studies

After the documentation listed above has been thoroughly reviewed, a field investigation of the structure should be conducted.

(1) General Survey - Overall dimensions and layout should be measured. Dimensions and details of structural members and connections should be verified wherever possible. Conformity with or deviation from original plans should be noted.

(2) Evidence of Distress - Any excessive deflection, cracking, crushing, distortion, and/or deterioration (e.g. corrosion) should be noted, measured (in so far as is possible), and recorded. Possible sources of distress should be identified wherever possible. Extent of repair should also be noted.

(3) Modification of Original Structure - Any additional penetrations (openings), closures (partitions), or strengthening of structures should be noted and verified against existing documentation. In the absence of such documentation, sufficient measurements should be

made to describe the effect of modifications on structural behavior.

(4) Contribution of Nonstructural Components to Structural Response - Connections between partitions and structural framing, and between exterior cladding and structural framing should be examined. The ability of these connections to develop adequate shear resistance should be evaluated. Where this cannot be done by observation and calculation, a laboratory test of a typical installation should be considered.

(5) Materials Sampling - Additional information on materials in-place may be obtained by nondestructive tests and by coring and laboratory testing of selected samples. This information should be used to modify field control data, thus reducing the uncertainty as to material characteristics to be used in analytical studies.

(6) Foundation Examination - Foundations should be examined for any evidence of settlement, poor drainage, and distress. Additional soils testing in the vicinity of a structure may be carried out if required for analytical studies.

(7) Special Investigations - For selected buildings, special on-site static loading tests may be carried out to determine stiffness or strength. In some cases, certain dynamic characteristics may be determined experimentally using vibration generators or ambient vibration recording devices.

(8) Potential Hazards of Mechanical, Electrical, and Other Services - The safety of an existing building as defined by hazard to life and the possibility of injury is usually viewed in terms of preventing structural damage and/or collapse. While it is true that the primary source of injury in an earthquake is the impact of falling objects, a number of other hazards exist. Failures in fuel systems may cause large fires, for example, and damage to fire control systems may further increase fire hazard. Other risks such as explosions, release of toxic chemicals, and disruption of disaster control services increase as a consequence of earthquakes.

In evaluating existing buildings for potential earthquake damage, it is necessary that service systems and nonstructural elements be evaluated to determine if they represent an added risk in the event of an earthquake. Elevators, mechanical and electrical equipment, and sprinkler and fire control systems must be designed, mounted, and braced to resist inertial forces, and must provide essential services after an earthquake. Suspended ceilings must be braced and mounted so as to prevent premature failure of mountings and thus collapse. Storage racks must meet similar design requirements to prevent contents from falling, particularly when toxic or flammable materials are being stored.

A more detailed description of essential service systems and special problems associated with protection of each follows:

Elevators - Cable guides and motor and counterweight mountings must resist inertial forces. An emergency source of power should be available. Elevator cages should be protected from falling objects.

Mechanical Equipment - Such equipment includes cooling towers, compressors, fans, pumps, boilers, furnaces, piping, air ducts, etc. Release of combustible fuel, toxic or high-temperature gases, and liquids that may be part of the mechanical equipment should be prevented.

Electrical Equipment and Lighting - Such equipment includes special motors, lighting fixtures, telecommunications systems, wiring circuits, and circuit control systems. Fail-safe devices should be provided in order to avoid short circuits that might initiate fires were any of these systems to fail. Suspended fixtures should be mounted or damped to minimize damage due to swaying or falling portions of lighting fixtures.

5C. Analytical Studies

The analytical method chosen to evaluate the structural safety of an existing building should incorporate the simplest mathematical model that yields sufficiently accurate results. In some cases, a seismic coefficient may suffice; in others, a modal response spectrum should be used to analyze the lateral force-resisting system of a structure. Occasionally, elastic analysis may be sufficient, while analysis of inelastic effects, including stiffness degradation and a complete time-history, may be required in selected cases. Detailed description of available analytical methods is beyond the scope of this paper. It should, however, be noted that for existing buildings, structural response analysis is more reliable than in the case of design, since geometric and material characteristics as well as details of elements and connection can be more reliably determined after construction.

Ground motion representing a selected level of damageability and risk for a given site is difficult to define. Where the importance of a building and of its safety justify the effort, however, special geoseismic studies of a particular site should be undertaken, leading to a more accurate description of ground motion than that prescribed for the region as a whole.

Evaluations of damageability and life safety have not been distinguished with adequate precision. Recent proposals for design requirements [5] recommend that structural performance be evaluated for two levels of lateral force than for the single maximum value V = KCW previously specified. Such dual earthquake criteria may be described as follows:

A moderate earthquake must be resisted without significant damage to structural elements and with the structure remaining essentially elastic. Such an earthquake should not result in any malfunction or damage of mechanical and electrical services (elevators, lighting, telecommunications systems, water supply, sprinkler, fire protection system, etc.), nor should contents of a building which may be hazardous to life or cause personal injury be damaged (e.g. release of toxic chemicals, fire, etc.).

A severe earthquake may produce significant local damage to structural elements without causing those elements to exceed permissible levels of ductility (deformation) and without collapse of significant portions of a structure. Such an earthquake may cause some life loss or personal injury, but the risk of these should be quite low. For different categories of buildings, various levels of damage to service systems and contents may be accepted depending on the role of structures in the community. For example, acceptable damage levels for disaster control centers, communication terminal facilities, etc., would be significantly lower than those for single-family dwellings, low-rise light industrial facilities, etc.

6. STRATEGY FOR HAZARD REDUCTION

As many as 200,000 potentially hazardous buildings exist in UBC seismic zone 3, with approximately half these buildings concentrated in the Los Angeles Metropolitan area. At the present rate of replacement, it may be fifty to one hundred years before the hazard due to these structures is eliminated. Any decision to reduce such hazard must take the form of public (government level) action. Legislation (or ordinances) requiring periodic safety evaluation of all buildings must be passed, establishing the principle of safety evaluation as a legal responsibility of ownership.

The extent and frequency of such evaluations would necessarily vary according to building type and category. For example, a hospital or school building might be required to undergo evaluation once every ten years, while residences would need to undergo evaluation only when ownership were transferred. Some inspection of residences is now required by sales agreements.

Appropriate regulation must include criteria for adequately evaluating safety and provide incentives for improving safety as well as penalties for maintaining hazardous conditions. Some steps in this direction have already been taken, e.g. City of Long Beach Ordinance [6]. Great care must be exercised, however, in assigning new legal responsibilities for building safety so that individuals and communities are not burdened by unreasonable economic hardship.

ACKNOWLEDGMENTS

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ASSESSMENT OF EARTHQUAKE SAFETY
AND OF HAZARD ABATEMENT

by

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ASSESSMENT OF EARTHQUAKE SAFETY AND OF HAZARD ABATEMENT

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SYNOPSIS

Methods for assessing the seismic safety of structures are discussed, and procedures for establishing priorities for evaluating and abating hazards are indicated. Field evaluation, code compliance evaluation, and maximum tolerable earthquake intensity evaluation are summarized, and results of a pilot study to identify possible hazards and levels of seismic resistance in several reinforced concrete frame buildings are reported.

1. INTRODUCTION

1.1 Need for Evaluation - The need for assessing the residual safety of buildings damaged in the event of a major earthquake is obvious. Immediate inspection of post-earthquake damage, under emergency conditions, is required to determine the condition of structures, the feasibility of occupying structures and resuming ordinary life processes of the community at an early date, and to determine which structures pose life or health hazards to the public and must therefore be demolished.

The need for evaluating potential seismic hazards in existing buildings is less obvious, but just as essential in regions of seismic activity. Most existing buildings were built before adequate seismic design standards were developed or accepted and these buildings may require some modification or strengthening to minimize the risk of injury or loss of life. If the same seismic performance criteria were used for existing buildings as are used for new buildings*, then clearly the same level of earthquake resistance must be developed in both old and new buildings. Even if the acceptable level of damage in existing buildings were greater than that for new buildings, existing buildings must be evaluated in order to determine which structures could be expected to sustain damage exceeding this level during an earthquake.

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Also, special hazards to the public may exist due to unsafe portions of buildings (usually nonstructural) such as ornaments, parapets, and accessways (stairs, elevators), which must be corrected. There are other conditions under which evaluation of existing buildings is essential. For example, structures damaged by nonseismic causes (e.g. fires, foundation distress, aging deterioration, corrosion, etc.) may have considerably less residual earthquake resistance than that provided in the original structure. Buildings which have undergone structural modification due to change in occupancy or for other reasons must also be evaluated in the modified state.

1.2 Evaluation Process - The process for evaluating the seismic safety (degree of hazard) of existing buildings in a given city requires the following two stages:

(1) Legal requirements must be established for reviewing seismic as well as other hazards in existing buildings and a judicial and administrative process instituted for carrying out this review.

(2) In order to evaluate the degree of hazard in the large inventory of existing buildings in a reasonable time and at

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* Buildings should: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, although some nonstructural damage may be allowed, and (3) resist major (severe) earthquakes without collapse, although some local structural damage may be allowed. Special public buildings should remain operational during and after the earthquake.

a reasonable cost, a systematic procedure for establishing priorities for review of classes of buildings and a methodology for evaluating hazards in individual buildings must be established. In some cases, review of design documents and a site inspection may be sufficient to determine the approximate degree of hazard in a given building. In other cases, more refined analytical evaluation may be required.

1.3 Priority Categories - Life safety and continuity of indispensable services are the bases for establishing priority categories. The following categories can be identified:

(1) Facilities which must remain operational during and after a severe earthquake.

(2) Essential institutions providing important social services which should continue to operate with minimal disruption.

(3) Buildings in which damage would result in high risk to life safety and concomitant disaster.

Other priority categories may be established on the basis of vulnerability associated with location (local seismicity), design standards of safety such as code requirements, workmanship, materials of construction, age, and possible deterioration. For example, the following categories may be identified:

(1) Buildings in high seismicity zones which were built prior to enforcement of the first effective seismic design provisions.

(2) Buildings in high seismicity zones which were built under old seismic design provisions, but which are constructed of unreinforced masonry, nonductile moment-resisting concrete frames, buildings with heavy precast concrete curtain walls or structural elements, and buildings of unusual construction or configuration.

2. EVALUATION METHODS

2.1 General - Various methods for evaluating hazards in classes of buildings and individual buildings, ranging from field evaluations which may require only a few man-hours to field-testing and sophisticated analyses which require thousands of man-hours, are available. Some of these methods are briefly reviewed here, and the results of a pilot study to identify possible hazards and levels of seismic resistance in several reinforced concrete frame buildings are reported.

2.2 Field Evaluation - Field evaluation methods rate a building rapidly and approximately as either "Good," "Fair" or "Poor" for a specified earthquake intensity. Review of design documents (drawings, calculations) and a site inspection should be sufficient for an appropriate rating. When plans and specifications for an existing structure are not available, field measurements, materials testing, and other means of identifying the construction scheme and the quality of materials and workmanship should be used. Basically, field evaluations determine whether or not a more detailed analysis of a building is necessary to assess its safety.

Several schemes for field evaluations have been proposed recently [1, 2, 3, 4]. Each of these schemes rates structures using a numerical or qualitative scale to evaluate a number of essential elements and characteristics of the buildings. The rating is then compared to a minimum composite score in order to classify the building.

The NBS field evaluation method (FEM) will be summarized here as representative of such methods. The first step in this method is to assemble information pertinent to determining the probable seismic performance of a structure. These data from an examination of plans and an on-site inspection are summarized in a standardized Data Collection Form. The geographic location of the building is assigned an expected Modified Mercalli Intensity (MMI), and the building is rated, as follows:

(1) Structural system general rating: (GR) is based on the type of structural system and construction materials. The rating

scale is from 1 to 4, with steel and ductile moment-resisting frames rated 1, and unreinforced masonry or unsheathed wood frames rated 4.

(2) Structural system vertical elements rating: (SR_1) is based on the quantity of resisting elements, symmetry of arrangement, and present condition. Each of these factors is rated on a scale from 1 to 4, and the composite score value of SR_1 is as follows:

$$SR_1 = \frac{1}{6}(Q + S) + \frac{2}{3} PC \qquad (1)$$

where Q is the quantity rating, S is the symmetry rating, and PC is the present condition rating.

(3) Structural system horizontal elements rating: (SR_2) is based on the worst case (largest grade on a scale from 1 to 4) of roof and floor rigidities (R), chord adequacy (C), and connections and anchorage (A), as follows:

$$SR_2 = \begin{array}{l}\text{largest values of A, C, or} \\ \text{R on scale from 1 to 4}\end{array} \qquad (2)$$

(4) Nonstructural systems are graded on a qualitative scale: Good (A), Fair (B), Poor (C), and Unknown (X). The principal items rated are:

a. corridor and stair enclosure walls (with regard to earthquake performance and life hazard),
b. interior partitions other than corridor and stair enclosures,
c. exterior curtain walls,
d. interior and exterior appendages, ornamentation,
e. ceiling and light fixtures,
f. glass breakage,
g. special hazards (gas connection, hazardous contents).

The overall composite rating: (CR) for the structural system is determined as follows:

$$CR = \frac{1}{3}[GR + 2(SR_m)]/ILF \qquad (3)$$

where SR_m is the larger values of SR_1 and SR_2, and ILF is the intensity level factor based on MMI varying from 1 to 4 as shown in Table 1. The structural system is then classified "Good" to "Very Poor," depending on the value of CR, as in Table 2.

The NBS field evaluation method has been used to evaluate a typical school building in California, resulting in a rating of "Good" for this building for MMI of IX. Results of other approximate methods of evaluating this building indicated that the risk of damage in a severe earthquake would be relatively high, and that more precise evaluations would be desirable.

While the NBS method is simple to apply, the results obtained by this method appear to be questionable. The algebraic formulations for SR_1 and CR_2 appear to be entirely arbitrary. The contribution of PC rating is given a 2/3 weight, whereas the other contributions are weighted at only 1/6 each. The present condition factor is given excessive weight, particularly for relatively new buildings, and the quantity (Q) of resisting elements is given too little weight. Furthermore, the strength of the building is not adequately accounted for.

2.3 Capacity Ratio - A possible measure of the seismic structural safety of an existing building is obtained by comparing its calculated earthquake resistance capacity to the design requirement for a similar new building. For this purpose, the structural system (geometry, materials, detailing) must be identified as completely as possible using design documents, site inspections, and testing. Then, using appropriate analytical techniques (the same as those used in designing new buildings), the value of required earthquake resistance, Q_{REQ}, must be determined on the basis of the element which is critical in resisting seismic effects. The available earthquake capacity, Q_{CAP}, for the same element must be determined using the criteria for evaluating capacity specified for designing new buildings. In the process of evaluating Q_{REQ} and Q_{CAP}, various modes of potential damage or failure must be considered and the critical element (or elements) must be identified. A measure of the earthquake safety of an existing building, relative to that of a new building, is defined by the capacity (or resistance) ratio R:

$$R = \frac{Q_{CAP}}{Q_{REQ}} \tag{4}$$

Depending on the desired level of performance, i.e. damage control or collapse control, the definitions of Q_{CAP} and Q_{REQ} may differ. In the case of damage control, these values should reflect the capacity of the weakest element in the building. In the case of collapse control, damage or failure of the weakest element may not result in collapse, as in highly indeterminate systems, and in such cases, Q_{CAP} and Q_{REQ} should be based on those critical elements which would initiate collapse in a progressive development of failure.

The capacity ratio R is an index of hazard: the lower the value of R, the greater is the hazard, potential damage, distress, and risk of collapse.

2.4 Code Compliance - When determination of Q_{REQ} and Q_{CAP} is based on the current code, this ratio may also be used as a measure of code compliance or noncompliance. The value of Q_{REQ} considers appropriate loading combinations with specified load factors, and the value of Q_{CAP} considers appropriate capacity reduction factors ϕ, as for example those given in the 1973 UBC or 1974 SEAOC. These load and capacity reduction factors may be either too high or too low for a given existing building, although their use is appropriate for designing new buildings. For buildings where previous damage or other deterioration has taken place, or for buildings where superior design and quality of workmanship has been observed, special ϕ factors should be used.

Determination of Q_{REQ} may be based on the response to the specified earthquake or on the response required to develop appropriate ductility in a flexural mode of failure. For example, using the 1974 SEAOC Recommendation for Seismic Design, the response to a specified earthquake may be expressed in terms of base shear Q_{REQ} as follows:

$$Q_{REQ} = Z\ I\ K\ C\ S\ W_E = C_E\ W_E \qquad (5)$$

where

Z - numerical coefficient related to the seismicity of a region

I - occupancy importance coefficient, varying from 1.0 to 1.5

K - numerical coefficient based on the dynamic response characteristics of the structure

C - numerical coefficient representing intensity and dynamic response characteristics of the building; variations in this coefficient in the building code standards during the past 70 years are shown in Table 3 (Ref. 5). The values shown in this Table indicate that the empirical expressions for C_E change from the simplistic conservative 1927 and 1935 UBC values, to more sophisticated and less conservative 1973 UBC values. However, more conservative values of C_E were proposed by the SEAOC in 1974, thus reversing the trend to lower values of C_E during the preceding thirty years.

S - numerical coefficient representing local site conditions, particularly site-structure interaction

W_E - effective weight of structure and other building components contributing to earthquake forces.

When Q_{REQ} is based on the condition that an element must not fail prematurely in a brittle mode, and that potential ductility of an element is fully developed, special code requirements for shear and moment capacities are specified. Such requirements were introduced in the SEAOC Recommendations in 1967 in Sec. 2630, Concrete Ductile Moment Resisting Space Frames. Thus,

$$Q_{REQ} = \frac{M_A + M_B}{L} + \alpha_3 Q_D + \alpha_4 Q_L \qquad (6)$$

where M_A and M_B are ultimate moment capacities of opposite sense at each end of the member, α_3 and α_4 are appropriate load factors (see Table 7), and subscripts D and L refer to dead and live loads, respectively.

2.5 Other Methods for Evaluating Safety - The degree of noncompliance with the current code expressed in terms of the capacity ratio R, Eq. 4, does not reflect the maximum earthquake resistance of existing buildings. This resistance may be expressed in terms of earthquake intensity, resulting in a specified degree of damage or failure.

If the maximum response of a structure, Q_{MAX}, can somehow be related to earthquake intensity and if the capacity of a structure, Q_{CAP}, is expressed in the same terms as the response, then the maximum tolerable earthquake will be such that:

$$Q_{MAX} = Q_{CAP} \tag{7}$$

Then, if a linear relationship between some measure of earthquake intensity and response is assumed, the earthquake intensity which will produce the specified degree of damage or failure can be determined. The maximum response can be defined as:

$$Q_{MAX} = Q_{GRV} + C_Q W_E \tag{8}$$

where Q_{GRV} is the effect of gravity loads, C_Q is a coefficient representing earthquake intensity, and W_E is the effective weight of the building. Then, the maximum value of the earthquake intensity coefficient, C_Q, which would result in maximum forces within the permissible limit Q_{CAP} is:

$$C_Q = \frac{Q_{CAP} - Q_{GRV}}{W_E} \tag{9}$$

The value of Q_{GRV} should consider effects of the deformed shape of the structure and of vertical accelerations under earthquake conditions. As a first order approximation, these may be neglected, and Q_{GRV} may be calculated on the basis of undeformed static conditions for (D+L) gravity loads. In this simple formulation of C_Q, the value indicates a base shear coefficient which can be related to earthquake intensity.

A variety of other methods for evaluating the structural adequacy of existing buildings may be used. Ideally, appropriate three-dimensional, nonlinear dynamic response analyses for different types and intensities of ground motion would provide the most reliable results. These analyses must account for soil-structure interaction and for the nonlinear behavior of structural elements under dynamic loading conditions. However, mathematical modeling of this problem is extremely complex, and available techniques are highly approximate. Therefore, the most desirable and practical method for evaluating structural safety would be one combining simplicity of execution with an acceptable level of reliability. Various methods are now being developed (Refs. 6-10) and their relative advantages can be determined by correlating results obtained by these methods in evaluating the response of relatively large groups of buildings.

3. HAZARD ABATEMENT

When for a given existing building the resistance ratio R, defined by Eq. 4, is equal to or greater than unity, it may be concluded that such a building complies with the current standards for seismic design of new buildings. However, when the calculated resistance ratio R is less than 1.0, the risk of earthquake damage in this building is larger than the risk of damage in a similar new building designed according to current standards. The degree of hazard indicated by R should be related to various risks, such as overall risk of life safety (e.g. life loss per 10^6 population per year), risk of life safety in buildings with high density occupancy, mix of the buildings in the community, risk of social and economic losses from interruption of services or use of special buildings and facilities (hospitals, fire service stations, communication centers, etc.).

A variety of options are available in hazard abatement:

(1) When hazard abatement is impossible or not economical, the building must be demolished.

(2) When preservation of the building and its use are essential, the building must be strengthened to an acceptable level of performance (R) within the required time.

(3) Intermediate corrective measures may include changes in use or occupancy, a reduction in the number of stories (partial demolition), or a reduction in projected lifetime (legal commitment to demolish within prescribed time limit).

(4) Acceptable combination of 2 and 3 above.

Because data are lacking for objectively correlating R values with various risks and for defining acceptable levels of hazard, decisions regarding hazard mitigation must be made on a subjective basis. Constraints on such subjective decisions must be derived on the basis of reasonable judgment, and on studies of probabilistic models of seismic damage consequences (hazards) and cost/benefit analysis.

For example, a subjective decision to accept a low value of R (say 0.10) may be rationalized for the existing inventory of buildings. In realistic terms, this subjective decision is based on accepting the principle that the earthquake safety of existing buildings will be improved through a natural process of "survival of the fittest."

On the other hand requiring uniform performance (risk of damage) for existing old and new buildings would necessitate upgrading all existing buildings to a value of R = 1.0, possibly involving considerable cost. Such expenditure may or may not be economically justifiable, except when special conditions require preservation of existing old buildings with a minimum risk of damage. When the cost of strengthening a building is not justified, the structure must be demolished or the larger risk of damage accepted.

An intermediate solution may be provided by varying acceptable values of R, depending on the nature and consequences of damage in different buildings. For example, critical or essential facilities which must remain operational during and after a severe earthquake should be strengthened to achieve a value of R = 1.0. Sufficient hazard abatement in other structures may be achieved using lower values of R.

The difference between the acceptable capacity ratio R and unity may be called the leniency ratio λ, so that

$$\lambda = (1 - R) \tag{10}$$

Different values of λ may be indicated for different categories of buildings. For example, it may be possible to establish building categories A, B, and C, specifying that $\lambda_A = 0.2$, $\lambda_B = 0.4$, and $\lambda_C = 0.6$.

For economic and technical reasons the objectives of hazard abatement in all existing buildings cannot be accomplished in a short period of time. For different categories of buildings the permissible time for compliance with hazard abatement requirements may vary from 15 to 35 years or possibly even longer periods of time.

The leniency ratios λ and the time duration for accomplishing the objectives of hazard abatement are closely related to social and economic considerations, such as acceptable risk levels (Ref. 11), capacity of the construction industry, availability of funds and rates of interest for financing hazard abatement, and economic incentives for investing in hazard abatement. A possible schedule for strengthening or demolishing hazardous buildings is illustrated in Table 4, where three categories of buildings are chosen in such a way that for Type A ($\lambda = 0.2$) all buildings will be brought up to capacity ratio $R = 0.8$ within 15 years, and for Types B and C ($\lambda = 0.4$ and 0.6, respectively) all buildings will be brought into compliance within 28 and 35 years, respectively. The schedule also accounts for the degree of hazard, so that buildings with lower capacity ratios R will be brought into compliance within a shorter time period (Fig. 1). For example, a building in Class B with a capacity ratio of $R = 0.2$ should be strengthened to $R = 0.6$ within 8 years or demolished. Another building in the same class but with $R = 0.4$ should be strengthened to $R = 0.6$ within 18 years.

In establishing building categories A, B, and C, the following factors may be considered: (1) use and occupancy of the building, (2) seismic zone and local site seismicity, (3) special hazards (release of toxic or combustible contents), (5) original design criteria (seismic intensity and seismic resistance, provisions considered in design), (6) original quality of materials and workmanship, and present physical condition (evidence of prior damage or deterioration).

The following classifications based only on use and occupancy may be adopted for a hazard abatement program. However, further refinements in these classifications may be introduced, considering factors other than use or occupancy.

Class A

Facilities which must remain operational during and after a severe earthquake

- Hospitals
- Police Stations
- Fire Stations
- Essential Communications
- Power Plants
- Water Plants

Class B

Other essential facilities

- Institutions
  - Incapacitated
  - Orphanages
  - Nursing Homes
  - Schools
  - Detention and Correctional
- Public Assembly
  - Schools
  - Theaters
  - Shopping Centers
  - High-Rise Buildings
- Hazardous Uses
  - Industrial (production)
  - Commercial (storage, service)
- Buildings in "Inner Fire Districts"

Class C

All buildings other than single- or two-family dwellings.

Other approaches to hazard abatement may involve "balanced risk" of damage or "cost effective" level of abatement. In both of these approaches, the "remaining life expectancy" of the building must be known. In practice, it is extremely difficult to ascertain this life expectancy.

In addition to the technical provisions for dealing with the criteria and methods for identifying the hazards and for their removal, legal and administrative procedures for a "just, equitable, and practical method" for hazard abatement must be included in the Code.

An important factor in implementing provisions for hazard abatement in existing buildings is capital investment. Normally, investment in new buildings or in other productive ventures is more profitable than investment in hazard abatement in existing buildings. Unless appropriate economic incentives are introduced for this investment, it may be very difficult to implement the requirements for hazard abatement, except through extensive demolition of old buildings, resulting in economic injury to owners and occupants as well as in social dislocations in the community.

4. EARTHQUAKE RESISTANCE OF TYPICAL BUILDINGS - PILOT STUDY

4.1 Introduction - A pilot study of the effect of building code changes on the earthquake resistance of low-rise reinforced concrete frame buildings was carried out and is briefly summarized below. The objective of this study was to calculate values of R (Eq. 4) and C_Q (Eq. 9) for typical 3- and 4-story reinforced concrete frame buildings designed in accordance with UBC Codes during the period 1946-1973. Computer programs were developed for generating building prototypes and for determining R and C_Q values for these prototypes.

In the evaluation, it was assumed that the critical element in a building frame was the beam-column joint at the first floor level, and that either a bending or shear mode of failure in either the beam or the column could control. The criteria for evaluation were the 1973 UBC and the 1974 SEAOC Recommended Lateral Force Requirements.

The principal variables were the number of stories (3 and 4), the material characteristics (f'_c = 3 ksi with f_y = 40 ksi, and f'_c = 5 ksi with f_y = 60 ksi), and the Code criteria used for design [UBC 1946, 1956, 1963 (WSD and USD), and 1973]. By combining different variables, twenty cases were studied. The following notation is used to describe the particular design: (Tables 5, 8, 9)

(N - number of stories) - (f'_c and f_y - concrete and steel strengths) - (Y - years)

Thus, 4-5-60-1964 refers to a 4-story building with 5 ksi concrete strength, 60 ksi reinforcing steel yield strength, designed in accordance with the 1946 UBC Code. For the 1963 designs, both the working stress design (WSD) and the ultimate strength design (USD) criteria were used.

A number of characteristics were held constant in designing the typical building elements.

Bay size:	25 ft. x 25 ft.	Floor dead load	= 100 psf
Floor System:	2-way slab	Floor live load	= 40 psf
Story Height:	12 ft.	Effective weight W_E	= 140 psf*
Beam width	= 12 inches	Column shape	square
Reinforcement ρ	= 0.0125†	Reinforcement ρ	= 0.035†
Concrete cover	= 2 inches	Concrete cover	= 2 inches
Stirrup steel f_y	= 40 ksi	Tie steel f_y	= 40 ksi

* includes weight of walls, partitions, and fixed equipment

† average value

The details of the connection are shown in Fig. 2 and are summarized in Table 5.

4.2 Frame Analysis Idealization - The response of the frame building was represented by that of an interior frame, and the ground story was considered to be the critical one. For gravity loads, it was assumed that the beams resist a maximum moment at the support $M_{GB} = (q_B L^2/11)$ and maximum shear $V_{GB} = (q_B L/2)$, where q_B is the gravity load per unit length of the beam, and L is the beam span (centerline dimensions). Under gravity loading, the column was assumed to resist axial load only, so that

$$N_{GL} = \Sigma_i \; p_i \; \ell_x \; \ell_y \tag{11}$$

where p_i is the combined dead and live load per unit area of the i^{th} story, and ℓ_x and ℓ_y define the contributing area for the column load. Possible live load reduction factors were neglected in this study.

For lateral loading, it was assumed that all inflection points were located at the midspan of the beams and at midheight of the columns. Furthermore, the overturning moment effect on axial load in the columns was neglected. Distribution of lateral loads is specified in the Code so that the column shear V_{EC} at the i^{th} story can be calculated and the column maximum bending moment M_{EC} is:

$$M_{EC} = \frac{1}{2} V_{EC} H_s \tag{12}$$

where H_s is the story height (centerline dimension).

The beam maximum moments were calculated assuming equal stiffness of the beams framing into the column, i.e. half the sum of the column moments above and below the beam level:

$$M_{EB} = \frac{1}{2} (M_{EC}^{i} + M_{EC}^{(i+1)}) \tag{13}$$

The beam maximum shear is then:

$$V_{EB} = (M_{EB}/0.5L) \tag{14}$$

Biaxial bending in the columns may occur when adjacent spans are not equal in both directions, or when both longitudinal and transverse earthquake components with respect to the building axes are considered. In this study, the effects of biaxial bending were neglected.

4.3 Forces Used in Design - The moments, shears, and axial forces in beam and column sections were calculated using the base shear force Q_{REQ} and the frame analysis idealization previously described. For buildings designed in accordance with WSD, a 0.75 reduction factor was used to evaluate the combined effect of gravity and earthquake, representing the permissible 0.33 increase in allowable stresses for this condition. For buildings designed in accordance with USD, appropriate load factors were used (Table 7).

The base shear force was calculated using Eq. 5 in which the coefficient C_E is specified in the appropriate Code. The values of C_E used in this study are shown in Table 6. The trend to lower values of C_E during 1946-1973 is clearly demonstrated. Also, the reversal of this trend in 1974 is shown.

General expressions for moment M, shear V, axial load N, in either beams or columns, can be written as a sum of the contributions due to dead, live, or earthquake loads with appropriate load factors. Two loading conditions were considered: gravity (G) only, and combined gravity and earthquake (G + E):

$$(M,V,N)_G = \alpha_1(M,V,N)_D + \alpha_2(M,V,N)_L \tag{15}$$

$$(M,V,N)_{G+E} = \alpha_3(M,V,N)_D + \alpha_4(M,V,N)_L + \alpha_5(M,V,N)_E \tag{16}$$

where α_i are the appropriate load factors specified in the codes. These factors are summarized in Table 7.

In order to ensure a ductile mode of failure, the 1967 SEAOC Recommendation specifies that the maximum shear force for USD should not be less than:

$$V_{G+E} = 1.4(V_D) + 1.4(V_L) + \frac{M_u^A + M_u^B}{L} \tag{17}$$

where M_u^A and M_u^B are the ultimate moment capacities of opposite sense at each end of the member, and L in this case is the clear length of the member. In 1973, this requirement was further clarified by stipulating that ultimate moment capacities M_u^A and M_u^B shall be computed with ϕ equal to 1.25 rather than 0.9 to allow for possible excess yield strength over the minimum specified value of f_y.

4.4 Design of Beams and Columns - In designing beams for bending compression, steel reinforcement was neglected and the reinforcement ratio ρ was taken approximately as 0.012. The beam width was taken as $b = 12$ in. for all cases, and the required depth d was calculated by equating moment resistance with maximum design moment. The beam-depth dimension was then rounded off to the nearest larger inch, and the number of bars was selected to provide the required area A_s as closely as possible using No. 8 bars. The beam was then checked for shear and shear reinforcement was provided in accordance with the relevant code requirements. In the older designs, No. 2 bars were used as stirrups, but in later designs No. 3 bars were used. In all cases, the yield strength of stirrup reinforcing steel was $f_y = 40$ ksi. The beam overall depth, tension steel reinforcement A_s, area of stirrups A_v, and their spacing s for all twenty cases are shown in Table 5.

The column design followed an iterative procedure with slightly different methods for estimating the initial column sizes for the WSD and and USD conditions. In both cases, the columns were taken as square in cross-section with lateral tie reinforcement. For 18 inches or smaller columns, 8 main bars were used, and for 20 inches or larger columns, 12 main bars were used. Bar sizes varied from No. 8 to No. 11. After initial column size and steel reinforcement were selected, the adequacy of the trail column was verified by constructing an appropriate interaction diagram (Fig. 3), and checking the design N and M values for compliance with the diagram.

Lateral ties were provided to conform to the minimum tie and shear reinforcement requirements. All ties were designed using No. 3 bar size and the tie arrangement shown in Fig. 3 was used. The column side dimension, total longitudinal steel reinforcement area A_{st}, the area A_v of lateral reinforcement effective in resisting shear, and the tie spacing S_c are shown in Table 5.

4.5 Discussion of Results - The values of the capacity ratio R and of the earthquake coefficient C_Q are summarized in Tables 8 and 9. Values of R below unity indicate that the particular element in the building does not have sufficient capacity to resist the earthquake intensity in a ductile manner as required by the current code. Four modes of failure were considered: beam bending and shear, and column bending and shear. However, all modes of failure which result in $R < 1.0$ indicate a deficiency in the required level of earthquake resistance.

It can be seen that for all buildings designed prior to 1967, when shear requirement to develop full moment capacity was introduced in the SEAOC Recommendations, column shear capacity is deficient. Capacity ratios for column shear for these old buildings vary from 0.2 to 0.5, and indicate a high degree of compliance.

The values of C_Q indicate the level of earthquake intensity which the particular existing building can resist without exceeding the capacity based on the specified code. In order to obtain a realistic estimate of the earthquake intensity coefficient C_Q, all load factors were taken as unity and all transverse steel reinforcement was assumed to resist shear, even when A_v was below the minimum value specified by the code.

Based on the capacity ratio values in Table 8, the maximum permissible time for hazard abatement was determined for building categories A, B, and C in accordance with the tentative schedule illustrated in Table 4. These values are shown in Table 10. It is interesting to note that in category C, none of the post-1946, 3-story reinforced concrete buildings need strengthening. For the 4-story buildings in this category, only pre-1963 buildings need some strengthening, and then only if their remaining service life is projected beyond 35 years (i.e., beyond the year 2010). In this case, strengthening would be required when buildings constructed in 1955 were to serve for a total of more than 55 years.

In category A, most 3- and 4-story reinforced concrete buildings would need strengthening in a relatively short period of time. Even most of the 1973 buildings would require strengthening within 12-14 years to comply with the 1974 SEAOC requirements with a leniency ratio of 0.2. In category B, a majority of the buildings in this pilot study would require strengthening within 18-28 years, i.e., when they reach a service age of 40-50 years.

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Table 1 Relationship of ILF to MMI

MMI	VIII+	VII	VI	V-
ILF	1	2	3	4

Table 2 Rating Classification vs. Composite Score

$CR < 1.0$	$1.0 \le CR \le 1.4$	$1.5 \le CR \le 2.0$	$2.0 < CR$
Good	Fair	Poor	Very Poor

TABLE 3 VARIATION OF SEISMIC COEFFICIENTS IN CALIFORNIA CODES

YEAR	EARTHQUAKE	UBC	SEAOC	CALIFORNIA STATE			CITY CODES		OTHER TOPICS ON EARTHQUAKE ENGINEERING
				PUBLIC SCHOOL BUILDINGS	GENERAL BUILDINGS	OTHERS	SAN FRANCISCO	LOS ANGELES	
1906	SAN FRANCISCO						06 30 PSF WIND LOAD *6		
							10 - 20 PSF WIND LOAD		
1920							20 - 15 PSF WIND LOAD		
23	(KANTO)								
25	SANTA BARBARA							25 SEISMIC DESIGN WAS ADOPTED IN SANTA BARBARA CITY CODE	25 US C&GS LAUNCHED STUDIES ON SEISMOLOGY
		27 EARTHQUAKE PROVISION IN APPENDIX: 0.075-0.1×(DL+LL) DEPENDING ON SOIL CONDITION				28 CHAMBER OF COMMERCE LAUNCHED STUDIES ON SEISMIC CODE			
1930									30 STRONG MOTION SEISMOGRAPH WAS DEVELOPED
									32 STRONG MOTION SEISMOGRAPHS WERE INSTALLED IN SO. CALIF.
33	LONG BEACH			33 FIELD ACT *2 MASONRY 0.1(DL+½LL) ALL OTHERS 0.02-0.05 × DL	33 RILEY ACT *3 0.02 × TOTAL DESIGN VERTICAL LOAD		33 0.02×TOTAL DESIGN *7 VERTICAL LOAD	33 0.08 × (DL+½LL)	33 STRONG MOTION WAS RECORDED IN LONG BEACH
		35 FOR ZONE 3 0.08-0.16(DL+1/2LL) DEPENDING ON SOIL CONDITION		37 0.06-0.10 FOR UP TO 3 STORIES WITHOUT MOMENT-RESISTING FRAME 0.02-0.06 FOR OVER 3 STORIES WITH MOMENT-RESISTING FRAME		39 CHAMBER OF COMMERCE PUBLISHED "BUILDING CODE FOR CALIFORNIA"			
1940				41 0.06-0.30 DEPENDING ON SOIL CONDITION					
				43 PUBLISHED IN TITLE 21, CALIFORNIA ADMINISTRATIVE CODE (CAC)				43 $C = \frac{0.6}{N+4.5}$ (N≤13) ONLY DL WAS CONSIDERED	
46	(NANKAIDO)	46 $C = \frac{0.6}{N+4.5}$ ONLY DL WAS CONSIDERED					47 C_{max} = 0.08(1 STORY) C_{min} = 0.037(30 STORIES) WITH VARIATION FOR SOIL CONDITION		
48	(FUKUI)		48 JOINT COMMITTEE (ASCE & SEAONC)						
1950			52 JOINT COMMITTEE REPORT C=K/T K=0.015 0.02≤C≤0.06 W = DL + 0.25LL	53 $C = \frac{0.6}{N+4.5}$	53 0.03 FOR HEIGHT < 40' *4 0.02 FOR HEIGHT > 40'	STATE BUILDING *5 STANDARD LAWS 53 STATE BUILDING STANDARD CODE WAS PUBLISHED IN TITLE 24 (CAC)	56 C=K/T K=0.02 0.035≤C≤0.075 W = DL+0.25LL	56 $C = \frac{0.0465}{N+0.9(S-8)}$ S=8 FOR N≤13, S=13 FOR N>13 HEIGHT LIMIT WAS REMOVED	56 I-WCEE
			59 V=ZKCW C=0.05/∛T T=0.05H/√D OR 0.1N F_t=0.1V HEIGHT >13 STORIES, >160 FT "MOMENT RESISTING SPACE FRAME" ADOPTED						
1960									60 II-WCEE
		61 V=ZKCW C=0.05/∛T T=0.05H/√D OR 0.1N F_t=0.1V "MOMENT RESISTING SPACE FRAME" ADOPTED					62 BEGAN TO FOLLOW SEAOC CODE THIS YEAR		
			63 13 STORY UPPER LIMIT DELETED		63 PROVISION OF LATERAL FORCE IN 1961 UBC & SEAOC WAS ADOPTED IN TITLE 24, CAC				63 ACI CODE REVISED
64	(NIIGATA)	64*							
			66 $F_t=0.004(h_n/D_s)^2$ "DUCTILE MOMENT RESISTING FRAME" ADOPTED	65 ESSENTIALLY SAME AS 1961 UBC				66 SAME AS 1964 UBC	
		67* $F_t=0.007V(h_n/D_s)^2$ "DUCTILE MOMENT RESISTING SPACE FRAME" ADOPTED							
68	(TOKACHI-OKI)		68*						
1970		70*							
71	SAN FERNANDO		71*	71 ESSENTIALLY SAME AS 1970 UBC	71 1970 UBC & SEAOC WERE ADOPTED IN TITLE 24, CAC			72 PROPOSED A NEW CONCEPT IN LATERAL LOAD REQUIREMENT	71 ACI CODE REVISED
		73* ZONING MAP WAS REVISED	73*					73 1970 UBC & SEAOC CODES FOR < 160 FT, DYNAMIC ANALYSIS FOR > 160 FT	
75			74 V=ZKSIW C=1/15√T S (SOIL) AND I (IMPORTANCE) FACTORS ADOPTED						

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* MINOR REVISION

*2 EDUCATION CODE SEC. 15451-15465, DIVISION OF ARCHITECTURE, STATE DEPT. OF PUBLIC WORKS IN CHARGE, OFFICE OF ARCHITECTURE AND CONSTRUCTION, DEPARTMENT OF GENERAL SERVICES SINCE 1965

*3 HEALTH & SAFETY CODE SEC. 19100-19171

*4 STATE BUILDING STANDARD CODE

*5 STATE BUILDING STANDARD COMMISSION IN CHARGE

*6 APPROXIMATELY EQUIVALENT TO C=1.0/D, D: DEPTH OF BUILDING

*7 SAME AS RILEY ACT

FROM REFERENCE 5

Table 4 Permissible Time for Hazard Abatement
(Time to strengthen or abolish, years)

Capacity Ratio R	Category A $\lambda = 0.2$	Category B $\lambda = 0.4$	Category C $\lambda = 0.6$
0.1	2	3	5
0.1-0.2	4	8	15
0.2-0.3	6	13	25
0.3-0.4	8	18	35
0.4-0.5	10	23	--
0.5-0.6	12	28	--
0.6-0.7	14	--	--
0.7-0.8	16	--	--

Table 5 Beam and Column Dimensions and Reinforcement Details
(See Fig. 2)

FRAME	YEAR	Beam (b = 12 in)				Column			
		H, IN	A_s, IN^2	A_v, IN^2	S_b, IN	t, IN	A_{st}, IN^2	A_v, IN^2	S_c, IN
1	46	32	4.8	0.10	10	22	18.7	0.44	11
	56	31	4.8	0.10	10	20	15.2	0.44	10
3-3-40	63W	29	4.0	0.22	12	18	10.2	0.37	9
	63U	27	4.0	0.22	12	18	10.2	0.37	9
	73	29	4.0	0.22	6	18	10.2	0.37	3
2	46	32	4.8	0.10	10	18	10.2	0.37	9
	56	31	4.8	0.10	10	18	10.2	0.37	9
3-5-60	63W	28	4.0	0.10	5	14	6.3	0.37	7
	63U	23	3.2	0.22	10	14	6.3	0.37	7
	73	24	3.2	0.22	5	14	6.3	0.37	4
3	46	35	4.8	0.10	10	24	18.7	0.44	12
	56	33	4.8	0.10	10	22	18.7	0.44	11
4-3-40	63W	30	4.8	0.22	12	20	15.2	0.44	10
	63U	28	4.0	0.22	12	20	12.0	0.44	10
	73	30	4.8	0.22	5	22	15.2	0.80	4
4	46	34	4.8	0.10	10	20	15.2	0.44	10
	56	33	4.8	0.10	10	18	12.5	0.37	9
4-5-60	63W	30	4.8	0.10	5	16	8.0	0.37	8
	63U	24	3.2	0.22	11	16	8.0	0.37	8
	73	25	4.0	0.22	4	16	8.0	0.37	3

Table 6 Base Shear Coefficients C_E

Year	Code	Number of Stories 3	4
1946	UCB	0.091(1)	0.091(1)
1956	UCB	0.080	0.072
1963	UCB	0.050	0.045
1973	UCB	0.050	0.045
1974	SEAOC	0.080	0.070(2)
1974	SEAOC	0.110	0.110(3)

(1) Base coefficient 0.080; C_E adjusted for 0.5 live load constribution to W_E; i.e., 0.080 (160/140) = 0.091.
(2) S = I = 1.0
(3) S = 1.5, I = 1.25

Table 7 Load Factors

Code	α_1	α_2	α_3	α_4	α_5
WSD	1.0	1.0	0.75	0.75	0.75
USD-63	1.5	1.8	1.25	1.25	1.25
USD-73	1.4	1.7	1.40	1.40	1.40

Table 8 Capacity Ratios R (Eq. 4) - 1973 UBC and 1974 SEAOC

1. Load factors and capacity reduction factors based on code
2. Shear resistance of reinforcement is neglected when $A_v < A_{vMIN}$

Frame	Year	R - UBC 1973				R - SEAOC 1974(1)			
		Beam		Column		Beam		Column	
		Bend'g	Shear	Bend'g	Shear	Bend'g	Shear	Bend'g	Shear
1	1946	1.44	0.51	3.15	0.41	0.96	0.51	1.34	0.41
	1956	1.39	0.50	2.41	0.46	0.92	0.50	1.03	0.46
3-3-40	1963W	1.09	0.81	1.61	0.59	0.72	0.81	0.69	0.59
	1963U	1.00	0.77	1.61	0.60	0.66	0.77	0.69	0.60
	1973	1.09	1.02	1.61	1.12	0.72	1.02	0.69	1.12
2	1946	2.19	0.54	2.25	0.45	1.45	0.54	0.96	0.45
	1956	2.11	0.53	2.25	0.45	1.40	0.53	0.96	0.45
3-5-60	1963W	1.59	0.81	1.23	0.71	1.05	0.81	0.52	0.71
	1963U	1.03	0.81	1.23	0.74	0.68	0.81	0.52	0.74
	1973	1.08	1.04	1.23	1.02	0.72	1.04	0.52	1.02
3	1946	1.46	0.53	3.07	0.36	0.87	0.53	1.19	0.36
	1956	1.36	0.52	2.78	0.37	0.81	0.52	1.08	0.37
4-3-40	1963W	1.22	0.77	2.15	0.46	0.72	0.77	0.84	0.46
	1963U	0.95	0.79	1.84	0.57	0.57	0.79	0.71	0.54
	1973	1.22	1.06	2.40	1.12	0.72	1.06	0.93	1.12
4	1946	2.14	0.55	3.30	0.34	1.27	0.55	1.18	0.34
	1956	2.07	0.55	2.34	0.38	1.23	0.55	0.91	0.38
4-5-60	1963W	1.85	0.78	1.54	0.54	1.10	0.78	0.60	0.54
	1963U	0.99	0.81	1.54	0.57	0.59	0.81	0.60	0.57
	1973	1.26	1.10	1.54	1.00	0.75	1.10	0.60	0.99

(1) In calculating C_E values, the following factors were used: S = 1.5, I = 1.25.

Table 9 Coefficient C_Q (Eq. 9) for Maximum Tolerable Earthquake

1. Capacity reduction factors based on UBC 1973 Code.
2. Load factors = 1.0.
3. Shear resistance of reinforcement is included in all cases.

Frame	Year	C_Q			
		Beam		Column	
		Bending	Shear	Bending	Shear
1	1946	.19	.15	.22	.16
	1956	.18	.14	.17	.13
3-3-40	1963W	.12	.18	.11	.10
	1963U	.10	.15	.11	.10
	1973	.12	.33	.11	.36
2	1946	.32	.24	.16	.13
	1956	.31	.22	.16	.13
3-5-60	1963W	.21	.25	.09	.08
	1963U	.11	.17	.09	.08
	1973	.12	.33	.09	.22
3	1946	.16	.14	.20	.14
	1956	.14	.12	.18	.12
4-30-40	1963W	.12	.14	.14	.10
	1963U	.08	.12	.12	.10
	1973	.12	.29	.15	.51
4	1946	.26	.19	.19	.12
	1956	.25	.18	.15	.10
4-5-60	1963W	.21	.21	.10	.08
	1963U	.09	.13	.10	.08
	1973	.13	.31	.10	.24

Table 10 Time for Abatement of Hazard in Different Building Categories

Hazard evaluation based on 1974 SEAOC values; see Table 8 for R values and Table 4 for permissible time for hazard abatement.

Building Category		A	B	C
Frame	Design Year	Col	Col	Col
3-3-40	1946	10	23	--
	1956	10	23	--
	1963W	12	28	--
	1963U	12	--	--
	1973	14*	--	--
3-5-60	1946	10	23	--
	1956	10	23	--
	1963W	12	28*	--
	1963U	12	28*	--
	1973	12*	28*	--
4-3-40	1946	8	18	35
	1956	8	18	35
	1963W	10	23	--
	1963U	12	28	--
	1973	--	--	--
4-5-60	1946	8	18	35
	1956	8	18	35
	1963W	12	28	--
	1963U	12	28	--
	1973	14*	--	--

*Note: Abatement not required by 1973 UBC.

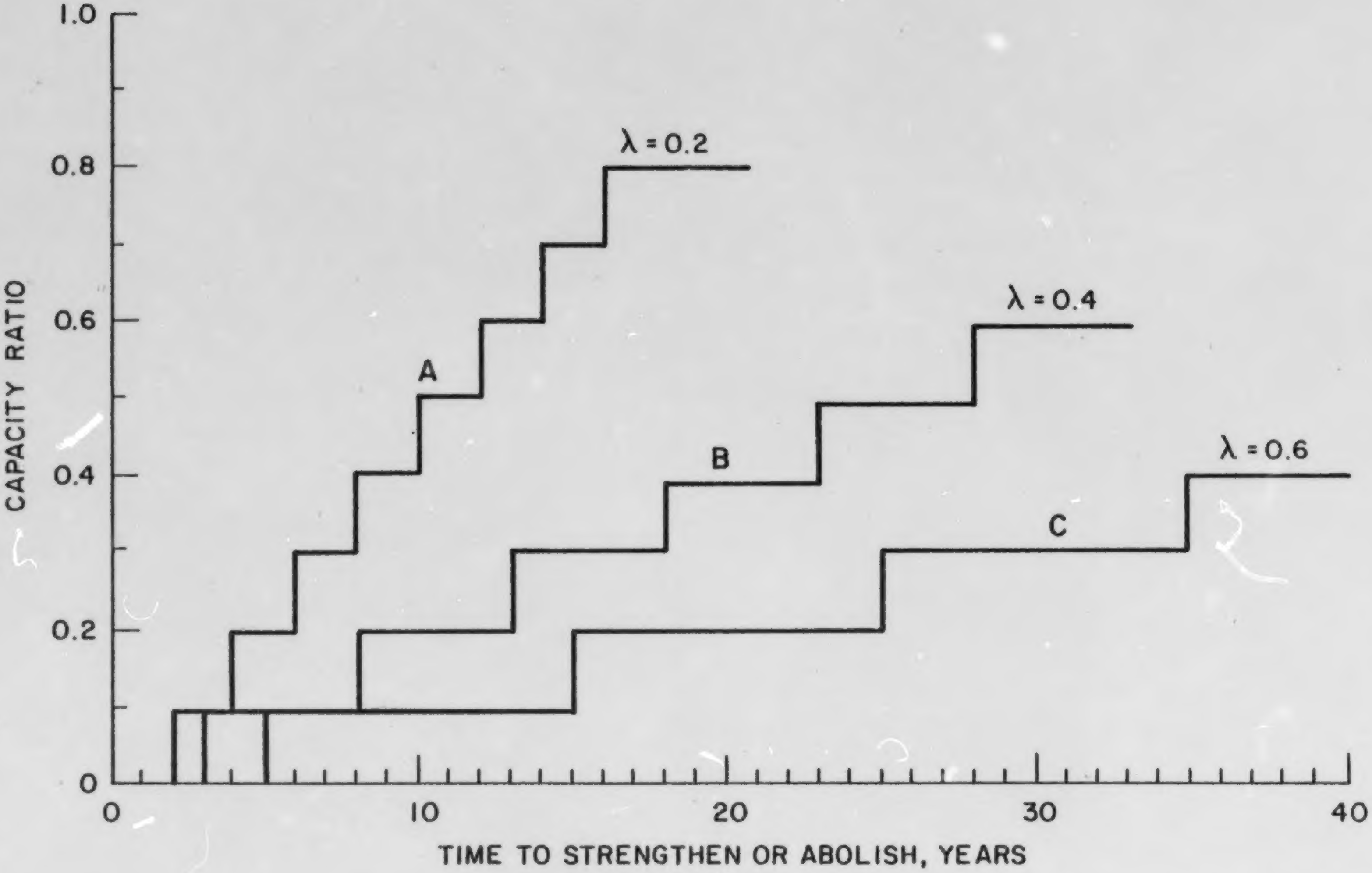

FIGURE 1 PERMISSIBLE TIME FOR HAZARD ABATEMENT

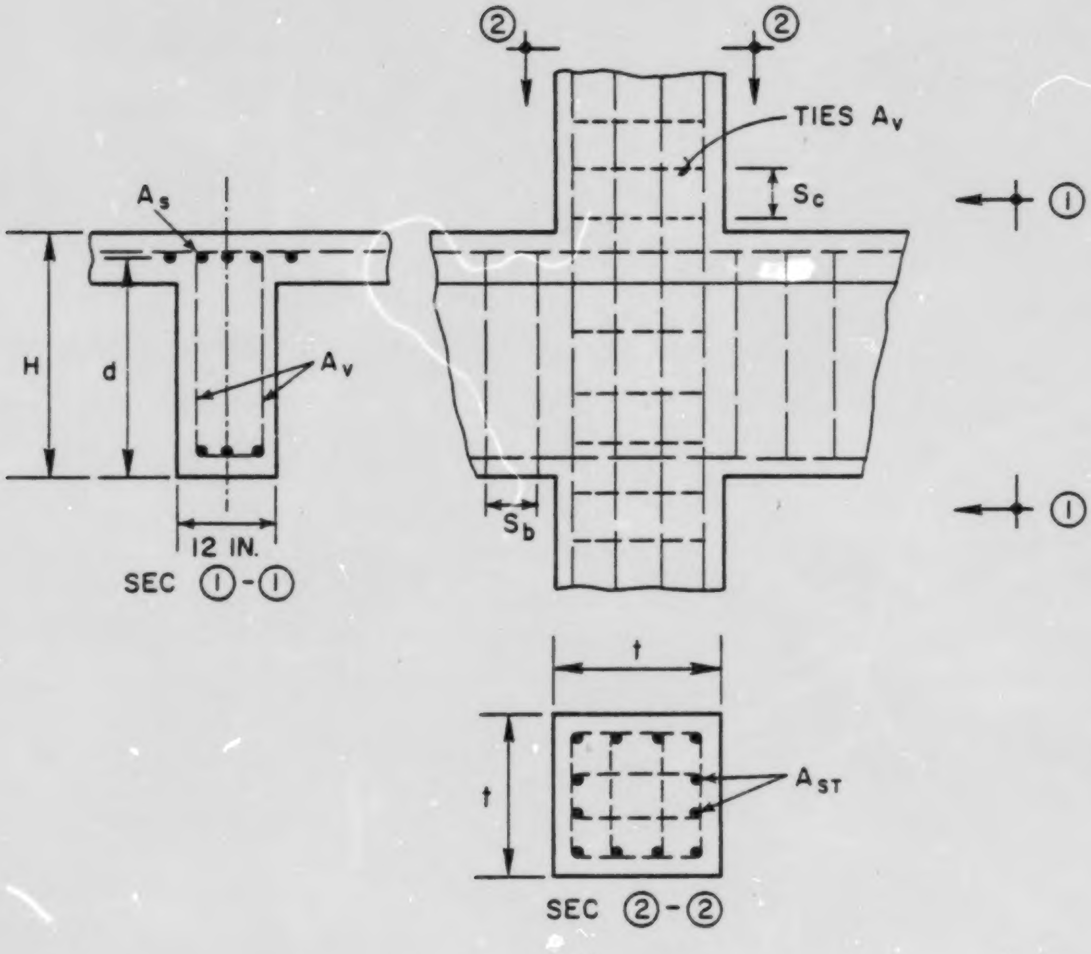
②
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FIGURE 2 TYPICAL DETAILS

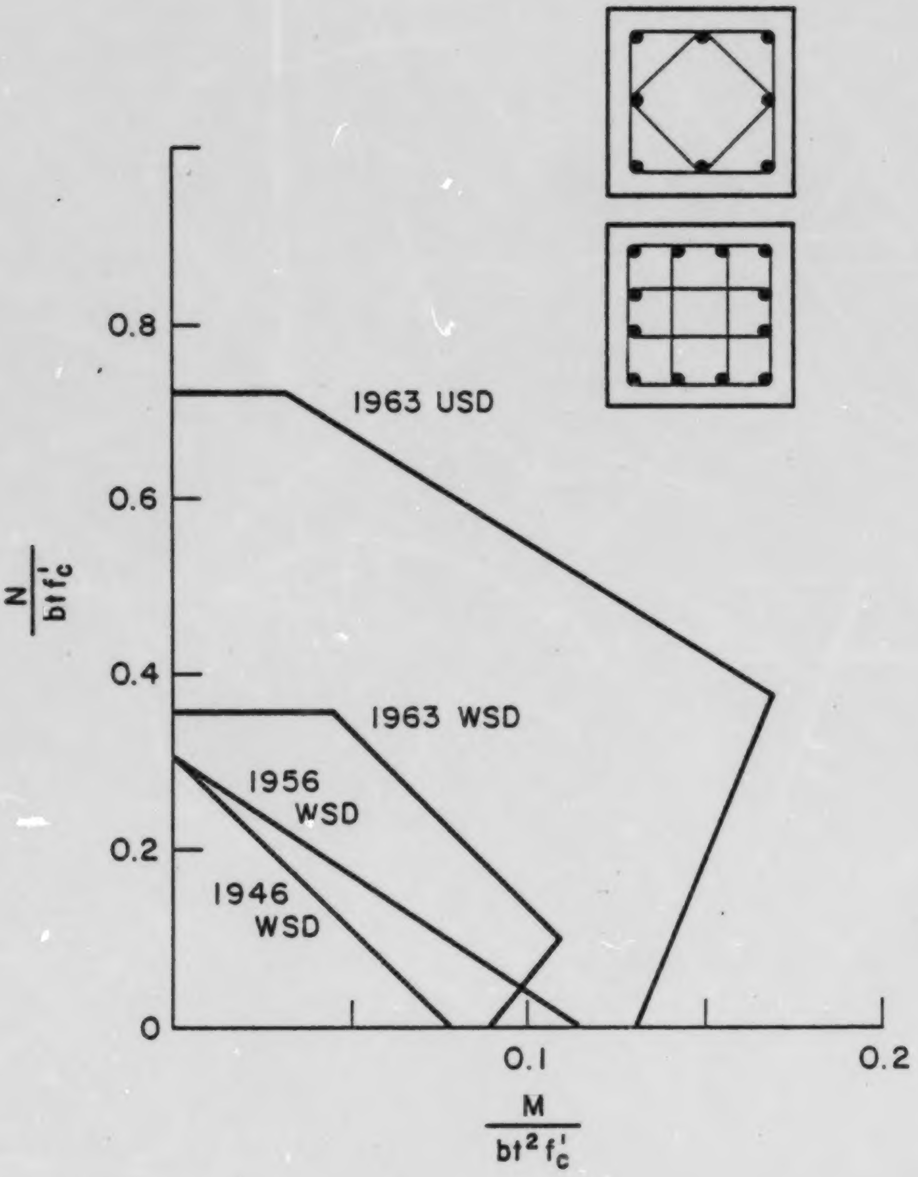

FIGURE 3 TYPICAL WSD AND USD INTERACTION DIAGRAMS FOR SQUARE COLUMNS

SEISMIC SAFETY OF
EXISTING LOW-RISE REINFORCED CONCRETE BUILDINGS
- SCREENING METHOD -

by

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Paper presented at a Review Meeting of the U.S.-Japan Cooperative Research Program in Earthquake Engineering with Emphasis on the Safety of School Buildings, Honolulu, Hawaii, August 18-20, 1975; published in the Proceedings of that Conference.

SEISMIC SAFETY OF
EXISTING LOW-RISE REINFORCED CONCRETE BUILDINGS
- SCREENING METHOD -

by

Tsuneo Okada[1] and Boris Bresler[2]

SYNOPSIS

This paper describes a methodology for evaluating the seismic safety of low-rise reinforced concrete buildings and its application to existing school buildings. The method classifies buildings according to three types of failure mechanisms; the criteria by which buildings are judged consider nonlinear behavior in response to two levels of earthquake motion. The overall method consists of a sequence of procedures which are repeated in successive cycles using more refined idealizations of behavior in each cycle. The first cycle of the procedure is called the "First Screening" and is the cycle described in this paper.

1. GENERAL

1.1 Introduction

A methodoloy has been developed for evaluating the structural adequacy of existing school buildings subjected to strong earthquakes [1]. In this paper, both the methodology and its application to the evaluation of existing school buildings are described. The method is based on the earthquake resistant design method for reinforced concrete buildings proposed by H. Umemura and others in 1973 [2,3]. However, as the method was initially developed for the design of new buildings, it has been revised and adapted especially for evaluating the structural safety of

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existing buildings. The method described here evaluates low-rise reinforced concrete buildings, but could, with appropriate modification, be applied to medium-rise reinforced concrete buildings.

Although the methodology presented here may require elaboration in the future, the basic concept of using criteria for evaluating structural safety and accounting for types of failure mechanisms and nonlinear behavior in response to graded earthquake motions will provide a basis for developing even more reliable methods of evaluation.

1.2 Screening Method

The structural safety evaluation considered in this report consists of a sequence of steps (Fig. 1), each following a procedure which will be described in Section 1.4. This procedure is repeated in successive cycles, the assumptions and details of the calculations being refined in each successive cycle when necessary for a reliable estimate of structural performance. This repetitive procedure is called "Screening," and is believed to be the fastest and the most practical method for reasonably evaluating the structural adequacy of a large number of buildings subjected to strong earthquake motions.

The first execution of the basic procedure is called the "First Screening." If a building cannot be classified as structurally safe after the first screening, a second more elaborate screening is required. The process continues until the structural adequacy (or inadequacy) of a building has been reliably estimated.

Three screening stages have been proposed in developing the methodology. In the first screening, the load-deflection characteristic of the first story or of the weakest story is approximately evaluated. This load-deflection characteristic is adopted as an analytical model and earthquake response is evaluated using linear response spectra for the strength safety evaluation and nonlinear earthquake response spectra for the ductility safety evaluation. In the second screening, the overall structural behavior of each story is estimated more precisely and a time history nonlinear response analysis is adopted. In the third screening, a nonlinear response analysis based on the nonlinearity of each member is adopted. Since the second and the third screening methods are not fully developed, this paper primarily describes the first screening.

1.3 Criteria for Evaluation of Structural Safety

The most important factors in determining structural adequacy are the criteria which define permissible damage resulting from a specified earthquake. The characteristics and intensities of future earthquakes are uncertain and the response of buildings to such earthquakes involves many unknown factors.

In attempting to account for these unknowns, two grades of earthquake ground motion and two degrees of building damage corresponding to the two ground motions were adopted as shown in Table 1(a). The decision criteria are based on the assumption that only slight structural damage which can be easily repaired is permitted for a strong earthquake, and that for a severe earthquake structural damage is permitted, but collapse is not.

1.4 Flow Diagram of Basic Procedures

A flow diagram of the procedure adopted in this report is shown in Fig. 1, which represents the procedure of the first screening; the procedure is basically the same for all screening stages, but the details of carrying out the calculations differ.

The procedure consists of the following five major steps:

(A) Structural Modeling
(B) Analytical Modeling (Evaluation of Structural Response under Lateral Forces)
(C) Strength Safety Evaluation
(D) Ductility Safety Evaluation
(E) Synthesis Evaluation of Safety

1.4.1 Structural Modeling - Step (A)

The evaluation is begun by selecting a structural model representing the load transmission system of the building. Gravity and seismic load transmission systems and the intensity of gravity load are determined by examining drawings, design calculations, specifications, construction records, and field investigations. Since proper structural modeling is one of the most important steps in evaluating structural safety, this step should be performed with utmost care. If, however, it is difficult to choose a structural model which accurately characterizes the structural behavior of the building, several types of models representing different load transmission systems should be investigated and the adequacy of the building should be judged within the bounds of the results based on the adopted structural models.

1.4.2 Analytical Modeling (Evaluation of Structural Response under Lateral Forces) - Step (B)

The load-deformation characteristics of a structural system subjected to lateral forces in both linear and nonlinear ranges are determined in this step. Analytical models for earthquake response analysis are also chosen.

1.4.3 Strength Safety Evaluation - Step (C)

The adequacy of lateral strength is evaluated by considering the relationship between the strength of the building and the applicable decision criteria. In order to ensure that only buildings having a high degree of seismic safety are classified as "safe," the strength requirement is evaluated using a linear earthquake response analysis. If it is not clear that a building fully satisfies the criteria matrix, it is classified as "uncertain," and the next step of the evaluation must be carried out. This step in the evaluation is used primarily in the first screening, because buildings which do not pass the first screening will probably be judged "uncertain" at this step in the second screening.

1.4.4 Ductility Safety Evaluation - Step (D)

The ductility safety evaluation is performed for buildings which are classified "uncertain" in the strength safety evaluation. This evaluation must be based on a nonlinear response analysis. If the response ductility of the building is greater than the specified limit value, then the building cannot be classified "safe" and a more precise evaluation of strength and ductility (the "Second Screening") must be carried out. If, however, it is clear that the building is "unsafe," the building is so judged at this step without requiring any further evaluation.

1.4.5 Synthesis Evaluation of Safety - Step (E)

While the question of seismic safety can be resolved in the previous step, it is recommended that the synthesis evaluation be performed as the final step of each screening stage, in order to determine how safe, unsafe, or uncertain a building may be. This step in the evaluation should also provide a basis for reviewing the many assumptions and unknowns incorporated into the screening process. The synthesis evaluation is helpful in indicating the need for rehabilitation and strengthening in existing buildings.

2. FIRST SCREENING METHOD

The criteria for evaluating structural safety and the procedure of the first screening method are described in this section.

2.1 Decision Criteria for First Screening

For the first screening, the terms "strong" and "severe" earthquakes and "reparable" and "noncollapse" structural damage are generally defined in Table 1(a) and are more precisely defined in Table 1(b). A strong earthquake was defined as having an intensity of 0.3g (i.e., 30% of gravity) and a severe earthquake as having an intensity of 0.45g, where intensity is given in terms of normalized peak ground acceleration.

Three different types of failure mechanisms, bending, shear, and shear bending, were considered. In a bending failure, the failure mechanism of the building is governed by the bending failure of members and the failure mechanism is ductile. In a shear failure, the failure mechanism of the building is governed by the shear failure of members and is not ductile but brittle. In a shear-bending failure, shear and bending failures in individual members occur with the possibility of shear cracking, but the overall failure mechanism is governed by bending.

The decision criteria are defined by considering the two earthquake intensities and the three types of failure mechanisms discussed above (Table 1(b)). This set of criteria is called the "Criteria Matrix."

The criteria are also illustrated schematically in Fig. 2, where the symbol ▼ indicates the criterion corresponding to each earthquake intensity and each type of failure mechanism.

The criteria matrix (Table 1(b)) together with the assumptions adopted in the analytical modeling define acceptable levels of damage for strong and severe earthquakes. The degree of damage acceptable in the event of a strong earthquake (0.3g) is defined to be less than that which occurred in buildings in the city of Hachinohe during the 1968 Tokachi-oki earthquake. For a severe earthquake (0.45g), a structure satisfying the criteria matrix must not collapse.

In order to improve the accuracy of the first screening, modifications of the criteria matrix should be made to account for the following:

(1) Local seismological conditions should be considered in choosing the intensity and characteristics of earthquake ground motion used in the evaluation.

(2) Since the ductility factors in the criteria matrix, i.e., 2.0 for an 0.3g earthquake or 4.0 for an 0.45g earthquake, are approximated for the overall ductility of buildings, these factors may be modified to account for the structural performance of a particular building. For example, if there is a sufficient amount of lateral reinforcement to ensure ductility greater than that defined by the criteria matrix, then the ductility factors of the criteria may be increased; if the axial stress in the column due to gravity load is large, the factor should be reduced.

(3) All buildings are classified into the three major types according to failure mechanism. However, if more failure mechanisms are considered, classification may result in more reliable evaluation. For example: (a) the mechanism governed by overturning of the foundation which is included in the bending type, and (b) the bending type of failure could

be subdivided into the beam yielding type and the column yielding type, because it is reasonable to allow higher ductility for the beam yielding than the column yielding type.

(4) As the criteria shown in Table 1(b) were defined for the overall response of a building, the matrix should be modified if the evaluation is based on the structural performance of each frame or each member.

These considerations are important for improving the reliability of the first screening method and in developing additional screening stages. Also, seismic safety may be reasonably evaluated if these considerations are accounted for by engineers when executing the proposed first screening.

2.2 Description of First Screening Method

The overall procedure of the first screening method is described in this section:

Step (A): Structural Modeling - The procedure for the first screening is the same as that for the general procedure described in Section 1.4.1.

Step (B): Analytical Modeling - Shear cracking strength, ultimate shear strength, and bending strength for all stories are calculated independently and the building is classified by failure type. Failure type is usually determined by the characteristics of the first story; if failure at another story controls, modification of the method is required [1].

By comparing the shear cracking strength C_{scl}, ultimate shear strength C_{sul}, and bending strength C_{Byl} in terms of base shear coefficients, the type of failure is determined as follows:

$C_{By1} < C_{sc1} < C_{su1}$: Bending type

$C_{sc1} < C_{su1} < C_{By1}$: Shear type

$C_{sc1} < C_{By1} < C_{su1}$: Shear-bending type

Load-deformation characteristics and the values in the decision criteria matrix also depend on the type of failure mechanism as shown in Fig. 2.

The fundamental natural period and modal participation factors are assumed either at this step or at the next step.

Step (C): Strength Safety Evaluation - The lateral strength determined at Step (B) is compared with the linear response base shear coefficients. If the building satisfies one of the following conditions, it is evaluated "safe" both for an 0.3g and an 0.45g earthquake:

For bending type : $C_E\ (0.3g) \leq C_{By1}$

For shear type : $C_E\ (0.3g) \leq C_{sc1}$ and $C_E\ (0.45g) \leq C_{su1}$

For shear-bending type: $C_E\ (0.3g) \leq C_{sc1}$ and $C_E\ (0.45g) \leq C_{By1}$

where

$C_E\ (0.3g)$ - Linear response base shear coefficient for 0.3g earthquake

$C_E\ (0.45g)$ - Linear response base shear coefficient for 0.45g earthquake

In this study, a standardized response spectrum was adopted for estimating linear response.

Step (D): Ductility Safety Evaluation - The first story response displacement is calculated using modified modal participation factors and a nonlinear response displacement spectrum; the safety of the building is then evaluated using this first story response displacement. If the response displacement of the first story is less than that defined by the criteria matrix, the building is evaluated "safe." The nonlinear response spectrum used in this evaluation must correspond to the type of failure mechanism established in Step (B). Therefore, three kinds of nonlinear response spectra corresponding to the types of failure mechanisms are used to evaluate response ductility [1].

Step (E): Synthesis Evaluation of Safety - The synthesis evaluation of safety in the first screening uses a shear strength-bending strength diagram with shear cracking strength and bending strength axes (Fig. 11(a)).

2.3 Details of First Screening Method

2.3.1 Step (A): Structural Modeling

The main items for the structural modeling are as follows:

(1) Structural System: The plan of each floor, section of each frame, cross-section of each member, and detailing of all joints are investigated through drawings. The foundation system should also be investigated by examining drawings and specifications. Any modification of the original design should be carefully checked by field inspection and all available documentation.

(2) Load Intensity: The average weight per unit floor area, including all gravity dead and live loads, is either determined from design calculations or independently calculated.

(3) Load Transmission System: Both gravity and seismic load transmission systems should be considered. A rough estimate of the building's safety may be made by an experienced investigator at this step.

(4) Properties of Materials: The specified material properties should be evaluated whenever possible. Information on soil conditions is necessary for evaluating the overturning capacity of the building, and should be ascertained from drawings or soil investigation reports.

(5) Design Method: Building code provisions, especially those adopted for the original seismic design, should be checked, and any discrepancy between design calculations and the code should be noted.

(6) Other Special Structural Features: Special features which might affect the seismic safety of a structure should be investigated. Such features include asymmetry and discontinuity in plan and in elevation, and local seismicity.

2.3.2 Step (B): Analytical Modeling (Evaluation of Structural Response under Lateral Forces)

The following approximations are adopted for estimating shear cracking strength, ultimate shear strength, bending strength, fundamental natural period, and modal participation factors:

(1) Shear Cracking Strength (C_{sci}): The average shear stress method (1,2) is used. If the shear cracking capacity of a story level is assumed as a function of the total cross-sectional area of concrete, then the shear cracking capacity can be determined as some assumed shear stress times the total area of concrete.

$$Q_{sci} = \tau_{av} \times (A_{ci} + A_{wi}) \tag{1}$$

where

Q_{sci} - shear cracking strength at i-th story

τ_{av} - assumed average shear cracking stress

A_{ci} - Σ column cross-sections at i-th story

A_{wi} - Σ wall cross-sections at i-th story

Defining the column-area ratio (a_{ci}) and the wall-area ratio (a_{wi}), the shear cracking strength in terms of shear coefficient (C_{sci}) is:

$$C_{sci} = \frac{Q_{sci}}{\sum_{j=i}^{n} \overline{W}_j} = \frac{\tau_{av}}{w_i} \times (a_{ci} + a_{wi}) \qquad (2)$$

where

$\overline{W}_j$ - weight of j-th story

n - total number of stories

a_{ci} - $\dfrac{A_{ci}}{\sum_{j=i}^{n} A_{fj}}$

a_{wi} - $\dfrac{A_{wi}}{\sum_{j=i}^{n} A_{fj}}$

A_{fj} - floor area of j-th story

w_i - average weight of the i-th floor level and above

$$\left(\sum_{j=i}^{n} \overline{W}_j \Big/ \sum_{j=1}^{n} A_{fj}\right)$$

If the average shear cracking stress τ_{av} is assumed, the shear cracking strength can be calculated by Eq. 2.

The average shear cracking stress is estimated by the following method:

Average shear stress when shear cracking occurs at the i-th seismic element of the j-th story is:

$$\tau_{av} = \tau_c \times \left(\frac{A_i}{\overline{A}_j} \Big/ \frac{K_i}{\overline{K}_j}\right) \qquad (3)$$

where

τ_c - shear cracking stress which was assumed as $4\sqrt{f'_c}$ (f'_c: concrete compressive strength in psi).

A_i - cross-sectional area of i-th element

$\overline{A}_j$ - Σ cross-sectional area of elements of j-th story

K_i - lateral stiffness of i-th element

$\overline{K}_j$ - Σ lateral stiffness of j-th story

The term $\left(\frac{A_i}{\overline{A}_j} \Big/ \frac{K_i}{\overline{K}_j}\right)$ in Eq. 3 is defined as the modification factor for shear cracking stress (α_s) and is assumed as follows:

If it is assumed that all wall elements and all column elements have similar geometries then the modification factor (α_s) for shear cracking in wall is obtained by:

$$\alpha_s = \frac{A_w}{A_w + A_c} \left(1 + \frac{K_c}{K_w}\right) \qquad (4)$$

where

A_w - Σ cross-sectional area of walls

A_c - Σ cross-sectional area of columns

K_c - Σ stiffness of columns

K_w - Σ stiffness of walls

The modification factor (α_s) can be approximately estimated by Eq. 4 by assuming the ratio (K_c/K_w).

(2) Ultimate Shear Strength (C_{sui}): Ultimate shear strength is calculated by the following equation:

$$C_{sui} = \alpha \times C_{sci} \qquad (5)$$

In the first screening, α is actually taken as 1.9. However, as this value has been derived from experimental data on shear walls surrounded by frames [2], it is recommended that the value of 1.9 be modified for walls without frames or for columns by considering shear span ratio, amount of shear reinforcement, etc.

(3) Bending Strength (C_{Byi}): Bending strength is evaluated by an approximate limit state analysis assuming that plastic hinges form at each connection of structural beams, columns, and footings.

The computer programs HMECH and SWALL have been developed for this purpose [1]. The base shear coefficient for a frame consisting of beams and columns is calculated by the following method:

At each connection, one of the following failure mechanisms is assumed: beam-hinge type, column-hinge type, or tie beam-footing type (Fig. 3). The type of mechanism assumed is determined by comparing either the sum of the column moments (above and below the connection) to the sum of the beam moments (to the left and right of the connection) or the column moment to the sum of the tie beam moments and the footing moment. The lowest sum determines the type of failure mechanism. The average moment for the type of failure mechanism is assigned either to the column above and below the connection or to the beam left and right of the connection. The shear force is then determined:

$$Q_i = \sum_{j=1}^{m} \frac{{}_T M_{cj} + {}_B M_{cj}}{h_i} \tag{6}$$

and

$$C_i = \frac{Q_i}{\sum_{j=i}^{n} \overline{W}_j} \tag{7}$$

where

Q_i - story shear at the i-th story

${}_T M_{cj}$ - moment at the top of the column

${}_B M_{cj}$ - moment at the bottom of the column

h_i - story height of the i-th story

m - number of columns and walls of the i-th story

C_i - shear coefficient at the i-th story

n - total number of stories

A shear wall with frames is modeled as an equivalent beam-column frame with rigid zones as shown in Fig. 4 and analyzed by the following method:

(1) Inflection points of the boundary beams and the tie beams are assumed between the midspan and the adjacent column line;

(2) Yield hinges at the end of the boundary beams are assumed to have formed;

(3) Distribution of lateral force is assumed to be either uniform or triangular along the stories;

(4) Base shear coefficients for all possible yield hinge mechanisms are calculated using equilibrium and the minimum value is used as the base shear coefficient.

The yield moments are calculated by the following equations [2,4]:

$$\underline{\text{Beam}}: \quad M_y = 0.9\, A_t f_y d \tag{8}$$

where

M_y - yielding moment

A_t - area of tension steel

f_y - yield strength of tension steel

d - distance from extreme compression fiber to centroid of tension steel

$$\underline{\text{Column}}: \quad M_y = 0.8\, A_t \cdot f_y \cdot D + 0.5ND \left(1 - \frac{N}{bDf'_c}\right) \tag{9}$$

If the axial load N is greater than $0.4\, bDf'_c$, this equation may not be used.

where

D - depth of column

b - width of column

N - axial load (positive in compression)

f'_c - compressive strength of concrete

Wall Surrounded by Columns: $M_y = A_g \cdot f_y \cdot L + \frac{N}{2} \cdot L$ (10)

where

A_g - area of longitudinal steel in a column

L - distance from the centroids of columns surrounding the wall

N - axial load (positive in compression)

Wall without Columns: Use strain compatibility or Eq. 9.

Footing: The moment based on soil-bearing capacity is substituted for the yielding moment of the footing.

$$M_y = \frac{f_o}{2} \left(1 - \frac{f_o}{f_b}\right) BL^2 \qquad (11)$$

where

f_o - stress of foundation soil by axial load $N(=N/BL)$

f_b - ultimate bearing stress of foundation soil

B - width of footing slab

L - depth of footing slab

(4) Natural Period: The following equation may be adopted for approximately estimating the fundamental natural period:

$$T = (0.06 - 0.10) \times n \qquad (12)$$

where

n - total number of stories

Generally speaking, a smaller value of T results in a conservative estimation of the nonlinear response displacement, but an unconservative estimation of the nonlinear response ductility factor. Therefore, it is recommended that a smaller value of T be assumed in calculating the response displacement for the ductility safety evaluation.

(5) Modal Participation Factor: The modal participation factors of the first mode are adopted, since the influence of higher modes is negligible for low-rise reinforced concrete buildings. An idealized lumped mass system, such as a system with uniformly distributed story masses and stiffnesses or a system with a linear mode shape, etc., is adopted for approximately estimating modal participation factors.

2.3.3 Step (C): Strength Safety Evaluation

In order to evaluate structural adequacy quickly, strength in terms of the base shear coefficient is compared to the linear response base shear coefficient. As shown in Fig. 5, if the linear response base shear coefficient falls within the range indicated by the heavy line, the building is considered to satisfy the decision criteria shown by the symbols ∇ and ▼, and is evaluated as "safe." Thus, as this evaluation primarily deals with strength, it is called the "Strength Safety Evaluation." Nonlinear response is indirectly considered in this step.

In calculating the linear response base shear coefficient C_E, the building is assumed to be a story level lumped mass system with n degrees of freedom (where n = no. of stories). The linear elastic response of the equivalent one-mass system is determined by assuming the first mode shape and neglecting the other modes. The response base shear coefficient, C_E, is then determined by the following equation:

$$C_E = \frac{\sum_{i=1}^{n} (\beta u)_i \ \overline{W}_i}{\sum_{i=1}^{n} \overline{W}_i} \times \frac{S_\alpha}{g} \qquad (13)$$

where

C_E - response base shear coefficient

$(\beta u)_i$ - modal participation factor at the i-th story

$\overline{W}_i$ - weight of the i-th story

n - total number of stories

S_α - linear response spectral acceleration

In calculating the linear response spectral acceleration S_α, it is desirable to use a response spectrum which considers foundation condition, local seismicity and other features at the site of the building. In order to simplify the evaluation, however, the following standardized spectrum by H. Umemura is adopted in this report.

$$S_\alpha = 3500 \cdot k_g \quad (\text{cm/sec}^2) \quad \text{for } T < 0.5 \text{ sec.}$$
$$S_\alpha = \frac{1750}{T} \cdot k_g \quad (\text{cm/sec}^2) \quad \text{for } T \geq 0.5 \text{ sec.} \qquad (14)$$

where

T - natural period of one-mass system in seconds

k_g - maximum acceleration of ground motion normalized by the acceleration of gravity g.

2.3.4 Step (D): Ductility Safety Evaluation

Step (D) estimates the first story displacement using nonlinear response spectra of displacement and modified modal participation factors to idealize the nonlinear behavior of the building.

The simple method adopted here roughly evaluates ductility. If, however, the result obtained using this method is questionable, the final evaluation of safety should be deferred.

In estimating building ductility:

1) the type of failure (type of hysteresis loop) is determined;
2) the equivalent one-mass system is estimated;
3) the normalized response spectrum is entered with an estimated natural period and strength of the equivalent one-mass system, and the maximum response ductility of the one-mass system (μ_0) is then estimated;
4) the response ductility factor at the first story of the building (${}_B\mu$) is estimated using μ_0 and the modification factor (m·f) for the modal participation factors; and
5) the ductility safety of the building is evaluated by comparing the response ductility factor (${}_B\mu$) with the decision criteria.

(1) Nonlinear Response Spectra: Nonlinear response displacement spectra for the Taft 1952, El Centro 1940 and Hachinohe 1968 earthquakes for the three types of hysteresis loops are used in the first screening.

They are:

Origin-oriented hysteresis loop for Shear type

Degrading Tri-linear hysteresis loop for Bending type

Modified Degrading Tri-linear hysteresis loop for Shear-Bending type.

The response spectra of the origin-oriented and the degrading tri-linear type are from Reference 2. The response spectra of the modified degrading tri-linear type were calculated by Dr. M. Murakami from Reference 1; two examples are shown in Fig. 13. The hysteresis loops are shown in Fig. 6.

(2) Equivalent One-Mass System and Modified Modal Participation Factors: A three-story shear type building is used to illustrate the procedure for assuming an equivalent one-mass system and for estimating the nonlinear response at the first story of the building (Fig. 7).

The basic assumptions for the procedure are that the first mode of vibration dominates in the linear range, and that each story reaches the critical stage simultaneously or the first story reaches the critical stage first.

The shear cracking strength of the equivalent one-mass system is:

$$k_c = C_{scl} \times \frac{\sum_{i=1}^{n} \bar{W}_i}{\sum_{i=1}^{n} (\beta u)_i \bar{W}_i} \qquad (15)$$

where

k_c - cracking strength in terms of shear coefficient of the equivalent one-mass system

C_{scl} - cracking strength in terms of shear coefficient of the first story of the building

$(\beta u)_i$ - modal participation factor of i-th story

$\bar{W}_i$ - weight of i-th story

For a low-rise building, the term, $\dfrac{\sum_{i=1}^{n} \bar{W}_i}{\sum_{i=1}^{n} (\beta u)_i \bar{W}_i}$ may be assumed as 1.0-1.2. The response displacement for the equivalent one-mass system obtained from the nonlinear response spectrum is modified for the ductility safety evaluation of the first story by the following method:

As shown in Fig. 7, the relationship between the displacement of the equivalent one-mass system and that of the first story of the multi-mass system is:

$$_B\delta_c = (\beta u)_1 \times \delta_c \tag{16}$$

$$_B\delta_{max} = (m{\cdot}f) \times (\beta u)_1 \times \delta_{max} \tag{17}$$

$$_B\mu_1 = (m{\cdot}f) \times \mu_o \tag{18}$$

where

$_B\delta_c$ - displacement at the first story of the building at the shear cracking stage

δ_c - displacement of the equivalent one-mass system at the shear cracking stage

$_B\delta_{max}$ - maximum displacement at the first story of the building

δ_{max} - maximum displacement of the equivalent one-mass system

$_B\mu_1$ - ductility factor at the first story of the building

μ_o - ductility factor of the equivalent one-mass system

$(m \cdot f)$ - modification factor

The modification factor $(m \cdot f)$ in Eqs. 17 and 18 is assumed considering the pseudo-modal participation which depends upon the mode shape in nonlinear range.

As shown in Fig. 7, if each story in Building Type A reaches the cracking stage simultaneously, the modification factor for displacement can be assumed as unity. The mode shape in the nonlinear range is assumed to be similar to the linear mode shape in this case.

In the case of Building Type B where the first story reaches the cracking stage before other stories, a modification factor should be adopted.

If it is assumed that the maximum displacement at the top of Building Type B is equal to that of Building Type A [5], the modification factor may be assumed as follows:

$$1 < (m \cdot f) < \frac{(\beta u)_{top}}{(\beta u)_1} \qquad (19)$$

where

$(\beta u)_{top}$ - modal participation factor at the top of linear system

$(\beta u)_1$ - modal participation factor at the first story of linear system.

2.3.5 Step (E): Synthesis Evaluation of Safety

The result of the first screening is illustrated on the shear cracking strength-bending strength diagram (Fig. 11).

This diagram is prepared as follows:

(1) Classification of the Type of Failure: Two lines are drawn on the shear cracking strength-bending strength diagram as shown in Fig. 8. The solid line indicates the boundary between the bending type and the shear-bending type and the broken line indicates the boundary between the shear-bending type and the shear type. The bending

strength and the shear cracking strength at the first story of the building obtained in Step (B) are plotted in this diagram.

(2) Zoning by Strength Safety Evaluation: Further zoning is possible both for an 0.3g earthquake and for an 0.45g earthquake by using the results of the strength safety evaluation (Step (C)) as shown in Fig. 9. C_E is the linear response shear coefficient at the first story from Eq. (13). The hatched zone shows that the safety of a building in this zone is uncertain at this step.

(3) Zoning by Ductility Safety Evaluation: The safety zone is enlarged by using the results of the ductility safety evaluation (Step (D)) as shown in Fig. 10. As the strength is adopted for the coordinates in Fig. 10, an appropriate conversion from displacement to strength is required to express the results of the ductility safety evaluation. For this purpose, a "Critical Strength" concept (2,3) is adopted in this report.

It has been recognized that the minimum strength which is required in order that a building's maximum response displacement be within the given ductility factor could be approximately estimated using a nonlinear response spectrum [2,6,7,8]. This minimum strength is called "Critical Strength." Generally speaking, critical strength depends on nonlinear load-deformation characteristics, damping characteristics, characteristics of the ground motion, etc.

In this report, these factors have been already assumed. Critical strength can be estimated if the natural period, the mode shape of the building, and the modification factor (m•f) for the mode shape in the nonlinear range are evaluated.

For example, for a bending type building in an 0.3g earthquake, the maximum allowable ductility factor of an equivalent one-mass system is:

$$\mu_0 = 2.0/(m \cdot f) \tag{20}$$

From the nonlinear response spectrum for the degrading tri-linear system, the minimum yield strength of the one-mass system (k_{cr}) for preventing larger displacements than the ductility factor of μ_0 is obtained as follows:

$$k_{cr} = \alpha_0 \cdot k_g \qquad (21)$$

The minimum yield strength of the first story is:

$$C_{cr} = \frac{\sum_{i=1}^{n} (\beta u)_i \bar{W}_i}{\sum_{i=1}^{n} \bar{W}_i} \times \alpha_0 \times k_g \qquad (22)$$

Similar considerations are possible for shear-bending and shear type buildings. However, since the critical strength of a shear-bending type building in an 0.45g earthquake depends on the ratio of bending strength and shear cracking strength, one critical strength which suffices for a number of buildings of this type cannot be defined. The boundary is, thus, neither parallel to the ordinate nor to the abscissa in Fig. 10, but is a curve beginning at point-1 and terminating at point-2 as shown in Fig. 10. In order to facilitate calculation and to keep the evaluation conservative, the line 1-2-3 was adopted instead of the curve 1-2 (Fig. 10).

In Fig. 10, C_{cr}(0.3g) and C_{cr}(0.45g) indicate the critical strengths for the 0.3g and 0.45g earthquakes. $\bar{C}_{cr}$ is the critical shear strength for the 0.45g earthquake.

Diagrams for the 0.3g and 0.45g earthquakes are shown together in Fig. 11(a) which is divided into nine zones. The characteristics of each zone are shown in Fig. 11(b). By plotting the results obtained by the first screening in a diagram such as Fig. 11(a), the synthesis evaluation of safety, including the ranking of safety, can be easily carried out.

The buildings belonging to Zones A, B, C, and D are evaluated "safe" in the Strength Safety Evaluation and are ranked as I. The buildings of Zone E are evaluated as "safe" in the Ductility Safety Evaluation and are ranked as II.

Because the buildings in Zones F and G satisfy either the criteria for an 0.3g or an 0.45g earthquake but not both, they are ranked as III. However, since they are located at the boundary between safety and unsafety, it is recommended that they be more precisely evaluated in further screenings.

The buildings in Zones H and I receive the worst ranking of IV. These buildings can be classified as "unsafe" in the first screening.

3. APPLICATION OF FIRST SCREENING TO EXISTING BUILDINGS

The method described above was applied to two school buildings in California; in this paper, these buildings will be identified as "School Building A" and "School Building B." The method was also applied to damaged and undamaged buildings located in the city of Hachinohe which was affected by the 1968 Tokachi-oki Earthquake.

3.1 School Building A

3.1.1 Step (A): Structural Modeling

(1) Structural System: School Building A, constructed in 1965, is a three-story reinforced concrete building consisting of core walls, precast concrete columns, and lift-slabs with post-tensioning. On the second and third floors there are exterior walls of precast concrete panels. The plan of the structural system is shown in Fig. 12.

(a) Foundation - Ground soil consists of "sandy silty clay." The allowable bearing capacities adopted in the original design were 3000 lb. per sq. ft. for the vertical load of (dead load + 1/4 x live load) and 4500 lb. per sq. ft. for (dead load + live load).

(b) First Floor - The first floor slab is a 4 in. concrete slab, directly supported on the ground soil. First floor vertical elements consist of precast concrete columns 16 in. x 16 in. with 4 No. 9 bars for exterior columns and 18 in. x 18 in. with 6 or 8 No. 9 bars for interior columns, core walls 9 in. thick, shear walls 10 in. thick, and brick veneer exterior walls. Since the brick veneer exterior walls are located at the columns' midspans and terminate at the ceiling, they are not considered to be structural elements.

(c) Second and Third Floors - The structural elements of the second and third floors are the same as those of the first floor with the exception of the reinforcement used for the interior columns and exterior walls. The floor slabs are concrete lift-slabs, 8-1/2 in. thick with post-tensioning. The slab is connected to columns by steel shear collars and shear bars inserted into the columns. The anchorage bars are placed at the connection between the slab and the concrete wall. The exterior wall was not considered to be a structural element in the original design. However, it is expected that the exterior wall would act as a structural element during an earthquake since lateral stiffness might be fairly great.

(d) Roof - The roof consists of roofing, vermiculite, and a post-tensioned concrete slab 8-1/2 in. thick.

(2) Load Intensity: The average dead weight of the building per unit floor area including beam, column, wall, and other dead load was calculated as 156 psf.

(3) Load Transmission System:

(a) The gravity load of the floor system is transmitted to the foundation primarily by the columns, although part of the gravity load can be transmitted through the interior walls. The exterior wall panels may also transmit some part of the gravity load.

(b) Seismic load is primarily transmitted through the core walls to the stairs and elevators and the walls in the F- and J-frames. These are called "core wall", "elevator wall," and "FJ-wall," respectively, in this paper. As there was a construction joint at the middle of the floor slab, the floor system of the building was considered to consist of two separate parts in the original design. But since the joint was filled by concrete after fabrication, the floor system was considered to be continuous in evaluating lateral force response. The exterior wall panels at the second and third floors were not considered to be structural elements in the original design, but can carry fairly large portions of lateral force.

(4) Material Properties: Material properties specified in the original design were as follows:

concrete - 5000 psi compressive strength for precast concrete columns

4000 psi lightweight aggregate concrete for slabs and walls

steel - A432 (Grade 60) for longitudinal reinforcement of columns

A15 (Grade 40) for other reinforcement

(5) Structural Design: The structural design of Building A was based on Title 19 and Title 21 of the California Administrative Code and the ACI Building Code (318-63). The adopted lateral shear coefficient was 0.092 for the first story, 0.109 for the second story, and 0.133 for the third story.

(6) Special Structural Features: In order to evaluate the behavior of this building in response to lateral forces, the following special features were considered:

(a) The stiffnesses of the slab-column connection and the slab-wall connection are uncertain; these values may significantly affect the lateral force capacity of the columns and walls.

(b) The strength of the slab, which could behave as an equivalent beam in the overall response to lateral forces, is uncertain.

(c) The stiffness and strength of the exterior precast concrete panels at the second and third stories are also uncertain.

In order to accommodate the range of values represented by these uncertainties, the following two structural models were adopted:

Model A: Lateral forces were assumed to be carried only by the core walls, elevator walls, and F-J walls.

Model B: Some part of the lateral force was assumed to be carried by the columns as well as the walls considered in Model A.

3.1.2 Step (b): Analytical Modeling

(1) Shear Cracking Strength: Shear cracking strength was evaluated using Eqs. 2 and 3. The wall ratio, column ratio, and wall-column ratio are shown in Table 2. Shear cracking strengths in terms of shear coefficients are shown in Table 3. In calculating shear cracking strength, the following values were assumed:

w = 172 psf including live load of 22.5 psf for the second and the third floors and 5 psf for the roof

τ_c = 280 psi (20 kg/cm^2) $(4\sqrt{f'_c},\ f'_c = 5000$ psi$)$

$\tau_{av} = \tau_c$ = 280 psi for Model A

$= 0.7\ \tau_c$ = 196 psi for Model B

In estimating τ_{av} for Model B, the modification factor α_s was calculated by Eq. 4 using the wall ratio and column ratio in Table 2 and assuming K_c/K_w to be 0.25.

Because it was predicted that shear cracking strength was greater than bending strength for Model B, it was not necessary to calculate ultimate shear strength.

(2) Bending Strength: Bending strengths in terms of shear coefficients are shown in Table 3. The computer programs HMECH and SWALL [1] were used in calculating the bending strength of frames and walls with boundary beams, respectively, based on the method described in Section 2.3.

The following assumptions were adopted in the calculation:

Yield strength of reinforcement: 60,000 psi for Grade 60 and 40,000 psi for Grade 40

Concrete compressive strength: 5000 psi for precast concrete columns and 4000 psi for walls

Ultimate bearing capacity of ground soil: A value twice the allowable bearing capacity of 4500 psf adopted in the original design was assumed

Bending capacity of equivalent beam for lift-slab: A value greater than the bending capacity of the columns in Model B was assumed.

(3) Estimation of Failure Type: In order to determine the failure type for the building, shear strength was compared to bending strength. For both Model A and Model B, the failure type was "Bending" as shown in Table 3.

In the case of Model A, the rotation of the footings of the shear wall, which is included in the "Bending Type" in this report, may govern the failure mechanism. In the case of Model B, the yielding of the columns as well as the rotation capacity of the shear wall may contribute to the failure mechanism. For buildings with such failure mechanisms, evaluations can be made for the first story.

(4) Fundamental Natural Period: A value of 0.3 sec. was assumed for the analysis using an approximation from Eq. 12.

(5) Modal Participation Factors: Assuming the uniform distribution of mass and stiffness, the modal participation factors were estimated as follows:

$$(\beta u)_3 = 1.22, \ (\beta u)_2 = 0.98, \ (\beta u)_1 = 0.54$$

3.1.3 Step (C): Strength Safety Evaluation

The linear base shear coefficient C_E was calculated using Eqs. 13 and 14.

For 0.3g earthquake: $C_E(0.3g) = 0.99$

For 0.45g earthquake: $C_E(0.45g) = 1.48$

After comparing the strength of the building shown in Table 3 with the linear response base shear coefficients, it was judged that the safety of the building could not be evaluated at this step.

3.1.4 Step (D): Ductility Safety Evaluation

The nonlinear response spectra for the degrading tri-linear loop shown in Fig. 13 were used for the ductility safety evalution.

(1) The strength of the equivalent one-mass system was calculated using Eq. 15 by substituting C_{By1} for C_{sc1}. The term $\left(\sum_{i=1}^{n} \bar{W}_i / \sum_{i=1}^{n} (\beta u)_i \bar{W}_i\right)$ was assumed to be 1.1 (Table 4).

(2) Nonlinear responses of the equivalent one-mass systems are shown in Tables 5(a) and 5(b). They were calculated by the following method:

> The X-direction of Model A for an 0.3g earthquake (Taft) is chosen as an example for explaining the method. Assuming a natural period of 0.3 seconds, the response displacement for a 1.0g earthquake was estimated as more than 12 inches (30 cm) (Fig. 13). The displacement of 30 cm was obtained from the curve for k_y/k_g of 0.5.
>
> The ductility factor was obtained by the following equation:

$$\mu = \frac{S_D \cdot k_g}{\frac{g}{4\pi^2} \cdot T_2^2 \cdot k_y} \qquad (23)$$

where

S_D - response displacement for 1.0g earthquake

T_2 - natural period for yielding stiffness

Substituting 30 cm for S_D, 0.37 for k_y/k_g, and $\sqrt{2}\ T_1$ ($\sqrt{2} \times 0.3$) for T_2, a ductility factor of 9 was obtained.

(3) The nonlinear responses of the building are shown in Table 6. They were obtained using the method described in Section 2.3.4, incorporating the response of the equivalent one-mass system. The modification factor (m·f) was assumed to be 1.0. This assumption is probably reasonable for Model A because the failure mechanism is governed by the rotation of the wall footing. However, this value is slightly unconservative for Model B because the failure mechanism in that case is a combination of the footing rotation and column yielding types. As shown in Table 6, the displacements of Model A are much greater than those allowed by the criteria, both for the 0.3g and 0.45g earthquakes. The displacements of Model B satisfy the criteria for all cases but that of the 0.45g earthquake of the 1968 Hachinohe EW type.

3.1.5 Synthesis Evaluation of Safety

The structural characteristics of the building are shown in Fig. 14. The critical strengths C_{cr} and $\bar{C}_{cr}$ were calculated using Eq. 22. From the response spectra for degrading tri-linear loop, the values of α_0 for C_{cr} were assumed to be 1.5 for an 0.3g earthquake and 1.0 for an 0.45g earthquake. For $\bar{C}_{cr}$, α_0 was assumed to be 1.5 for an 0.45g earthquake from the response spectra for an origin-oriented loop.

The results of the safety evaluation described above suggest the following:

(1) If Model A represents the building, the extremely large displacement beyond the displacement capacity may occur in both the 0.3g and 0.45g earthquakes. The building is thus evaluated to be "unsafe".

(2) If Model B represents the building, the building may be "safe" in an earthquake of the Taft 1952 type or the El Centro 1940 type, but "uncertain" in an earthquake of the Hachinohe 1968 type.

For Model B, it was assumed that the bending moment of the column transferred fully to the slab through the joint, while the moment transmission through the joint was neglected for Model A. Considering the detailing of the joint, the real behavior of the building may be supposed to lie between that of Model A and Model B, but closer to Model A.

The final decision as to the safety of the building at the end of the first screening was that it was "uncertain," but close to "unsafe."

3.2 School Building B

School Building B, constructed in 1964, is a two-story reinforced concrete structure with a partial basement, consisting of beams, columns, joist slabs, and tilt-up concrete walls as shown in Fig. 15.

The gravity load of the floor system is transmitted primarily through the beams in the Y-direction and the columns to the foundation. Some part of the gravity load may be carried by the walls. The seismic load is transmitted through the columns and the walls. However, the lateral force transmission capacity of the walls in the X-direction is uncertain because the stiffness of the joint between the wall and the slab is not known. A base shear coefficient of 0.133 was adopted for the original seismic design.

Concrete with compressive strengths of 2500, 3000, and 2000 psi was used for the frames, walls, and footings, respectively. A15 steel (Grade 40) was used as reinforcement.

Since the stiffness of the joint between the slab and the wall in the X-direction (walls in lines 2 and 5 in Fig. 15) was not known, two structural models were adopted for the X-direction. In one model, Model XA, the walls mentioned above were not considered to be seismic elements, and in the other, Model XB, the contribution of such walls to the lateral force capacity was fully considered. The fundamental natural period was assumed as 0.2 sec. for the Y-direction and for Model-XA, and as 0.16 sec. for Model-XB.

The response displacement and ductility factor of the building are shown in Table 7 and the characteristics of the building are shown in Fig. 16. The failure mechanism in the Y-direction is estimated as "Bending Type" and the building is evaluated as "safe."

The safety of the X-direction strongly depends on the behavior of the exterior precast concrete tilt-up walls in lines 2 and 5. If the stiffness and strength of the joint between the slab and the wall were enough to transfer shear force, then the failure type in the X-direction would be "Shear Type" and the building would be evaluated as "safe." If, however, the stiffness and strength of the joint were insufficient, a large displacement would be predicted and the building might be judged "unsafe." More investigation of the detailing of the joint is required.

As far as can be determined from the drawings, it would not be difficult to increase the stiffness and strength of the building even if structural performance at the joint were evaluated to be adequate.

3.3 Building in the City of Hachinohe in the 1968 Tokachi-oki Earthquake

The characteristics of the reinforced concrete low-rise buildings in the city of Hachinohe during the 1968 Tokachi-oki Earthquake are shown in Fig. 17.

The major assumptions adopted in the evaluation were:

Average weight of the buildings: 1 t/m^2 (205 psf)

Average shear cracking stress: $\tau_{av} = 10\ kg/cm^2$ (140 psi)

In estimating C_{cr} using Eq. 22, the term:

$$\frac{\sum_{i=1}^{n} (\beta u)_i \bar{W}_i}{\sum_{i=1}^{n} \bar{W}_i} \times \alpha_0$$

was assumed to be 1.5 for an 0.3g earthquake and 1.0 for an 0.45g earthquake.

It should be noted that the proposed first screening method can evaluate buildings damaged in an earthquake.

ACKNOWLEDGMENTS

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TABLE 1 CRITERIA MATRIX FOR JUDGING EARTHQUAKE SAFETY OF REINFORCED CONCRETE BUILDINGS

(a) General Criteria

Grade of Earthquake	Strong Earthquake	Severe Earthquake
Grade of Safety	Reparable Damage	Noncollapse

(b) Criteria for First Screening Stage

Failure Mechanism	0.3g Earthquake	0.45g Earthquake
Bending Type (Ductile)	Ductility Factor (μ)1) is less than 2.0	Ductility Factor (μ) is less than 4.0
Shear Type (Brittle)	Shear cracking stage	Before shear failure stage2)
Shear-Bending Type	Shear cracking stage	Yielding stage3)

1) ductility factor = maximum displacement/yield displacement.

2) shear deformation at this stage is considered to be one-half of the ultimate deformation capacity ($\gamma_{ult} = 4 \times 10^{-3}$ radian).

3) displacement at this stage is considered to correspond approximately to a ductility factor of 2.0 for the bending type.

TABLE 2 WALL RATIO, COLUMN RATIO, AND WALL-COLUMN RATIO OF SCHOOL BUILDING A

	Story	Wall Ratio a_w (in^2/ft^2)	Column Ratio a_c	Wall-Column $a_{wc}=a_w + a_c$
x-direction	3	0.60	0.49	1.09
	2	0.30	0.24	0.54
	1	0.20	0.16	0.36
y-direction	3	0.38	0.49	0.87
	2	0.21	0.24	0.45
	1	0.21	0.16	0.37

TABLE 3 STRENGTH IN TERMS OF SHEAR COEFFICIENTS OF SCHOOL BUILDING A

x-Direction

	Model A		Model B	
	Shear* C_{sci}	Bending C_{Byi}	Shear* C_{sci}	Bending C_{Byi}
3	1.0	0.18	1.26	0.55
2	0.52	0.13	0.63	0.34
1	0.33	0.10	0.41	0.27

*Shear cracking strength

y-Direction

	Model A		Model B	
	Shear* C_{sci}	Bending C_{Byi}	Shear* C_{sci}	Bending C_{Byi}
3	0.62	0.30	0.99	0.62
2	0.34	0.22	0.52	0.40
1	0.34	0.17	0.42	0.32

*Shear cracking strength

TABLE 4 STRENGTH OF EQUIVALENT ONE-MASS SYSTEM FOR SCHOOL BUILDING A

Earthquake k_g		Model A			Model B		
		Strength		k_y/k_g	Strength		K_y/k_g
		Building ${}_BC_y$	One-mass k_y		Building ${}_BC_y$	One-mass k_y	
0.3	x	0.10	0.11	0.37	0.27	0.30	1.00
	y	0.17	0.19	0.63	0.32	0.35	1.17
0.45	x	0.10	0.11	0.24	0.27	0.30	0.67
	y	0.17	0.19	0.42	0.32	0.35	0.78

TABLE 5(a) RESPONSE DISPLACEMENT OF EQUIVALENT ONE-MASS SYSTEM FOR MODEL A: 1.0g EARTHQUAKE

		Response Displacement in inches or in (cm)		Response Ductility Factor			
	k_y/k_g	Taft 1952(EW)	Hachinohe 1968(EW)	Taft 1952(EW)	Hachinohe 1968(EW)	Hachinohe 1968(NS)	El Centro 1940(NS)
x	0.37 (for 0.3g)	>12 in. (>30) cm.	>40 (>100)	>9	>30	--	--
	0.24 (for 0.45g)	>>12 (>>30)	> 40 (>>100)	>>14	>>45	--	--
y	0.63 (for 0.3g)	8 (20)	40 (100)	3.6-7	18-36	--	--
	0.42 (for 0.45g)	12 (30)	>40 (>100)	8-16	> 27	--	--

TABLE 5(b) RESPONSE DISPLACEMENT OF EQUIVALENT ONE-MASS SYSTEM FOR MODEL B: 1.0g EARTHQUAKE

		Response Displacement in inches or in (cm)		Response Ductility Factor			
	k_y/k_g	Taft 1952(EW)	Hachinohe 1968(EW)	Taft 1952(EW)	Hachinohe 1968(EW)	Hachinohe 1968(NS)	El Centro 1940(NS)
x	1.0 (for 0.3g)	6 (15)	8 (20)	1.7-3.5	2-5	1-2	2-4
	0.7 (for 0.45g)	6-8 (15-20)	28-40 (70-100)	2.5-6	10-30	--	--
y	1.2 (for 0.3g)	5 (12)	6 (15)	1-2	1.5-3	1-1.5	1.5-3
	0.8 (for 0.45g)	6 (15)	20 (50)	2-4	7-14	--	--

TABLE 6 RESPONSE DISPLACEMENT AT FIRST STORY OF SCHOOL BUILDING A

			Response Displacement in inches		Response Ductility Factor			
Model	Direction	k_g	Taft 1952(EW)	Hachinohe 1968(EW)	Taft 1952(EW)	Hachinohe 1968(EW)	Hachinohe 1968(NS)	El Centro 1940(NS)
Model A	x	0.3g	>2	>6.5	>9	>30	--	--
		0.45g	>>3	>>10	>>14	>>45	--	--
	y	0.3g	1.3	6.5	4-7	18-36	--	--
		0.45g	3	>10	8-16	>27	--	--
Model B	x	0.3g	1	1.3	1.7-3.5	2-5	1-2	2-4
		0.45g	1.5-2	7-10	2.5-6	10-30	--	--
	y	0.3g	0.8	1	1-2	1.5-3	1-1.5	1.5-3
		0.45g	1.5	5	2-4	7-14	--	--

TABLE 7 RESPONSE DISPLACEMENT AT FIRST STORY OF SCHOOL BUILDING B

		Response Displacement in inches		Response Ductility Factor			
Earthquake	Direction (model)	Taft (EW)	Hachinohe (EW)	Taft (EW)	Hachinohe (EW)	Hachinohe (NS)	El Centro (NS)
0.3g	x (model XA)	1.3-1.5	5-6	5-13	20-45	--	--
	x (model XB)	0.2	0.1	1.0	0.8	0.5	0.8
	y	0.4	0.2	≑ 1	1	1	≑ 1
0.45g	x (model XA)	2.6-3.9	10-17	12-32	43-136	--	--
	x (model XB)	0.6	0.2	5.5	1.0	1.0	3.5
	y	0.75	0.6-0.9	1-2	1.0	≑ 1	1.5-2

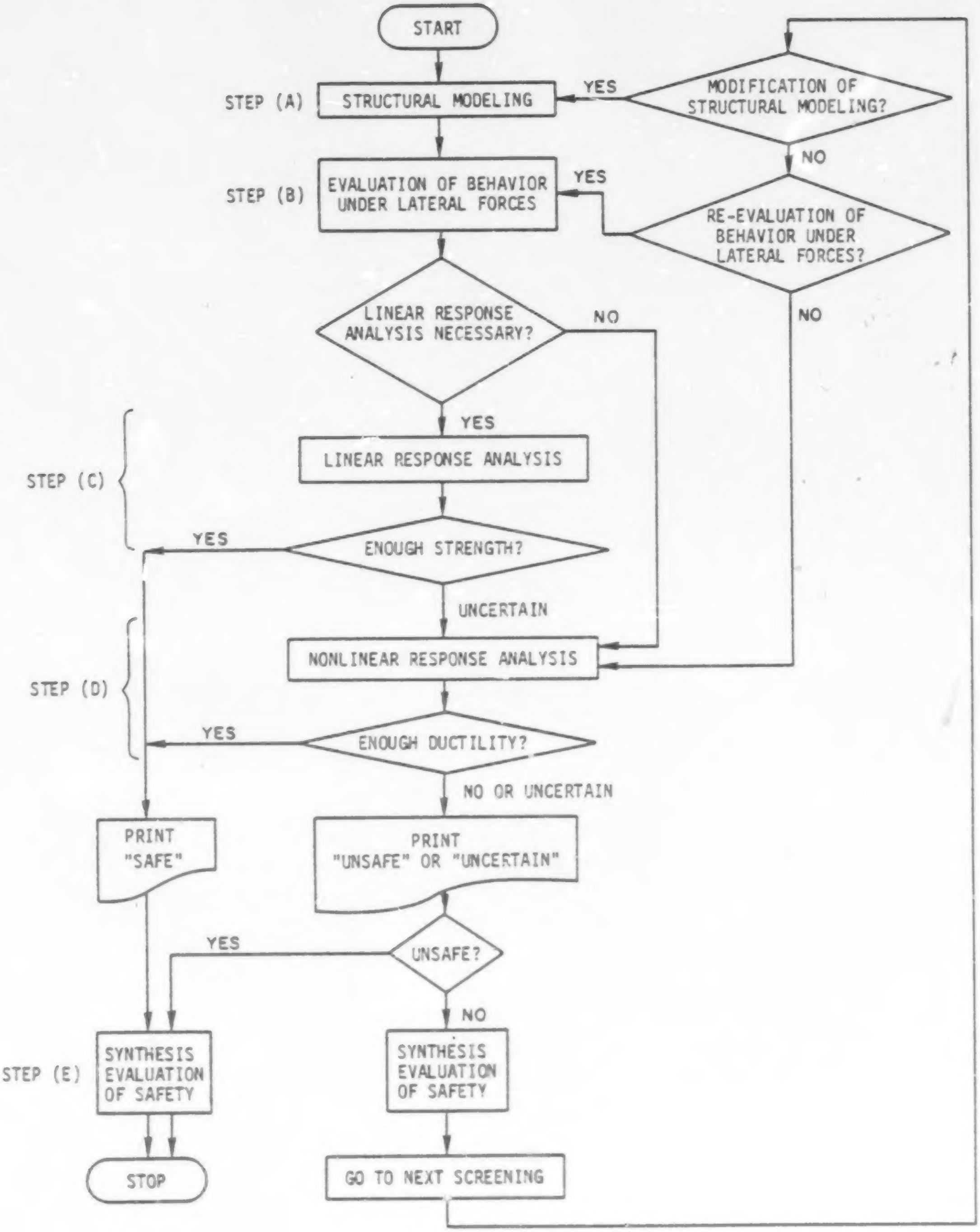
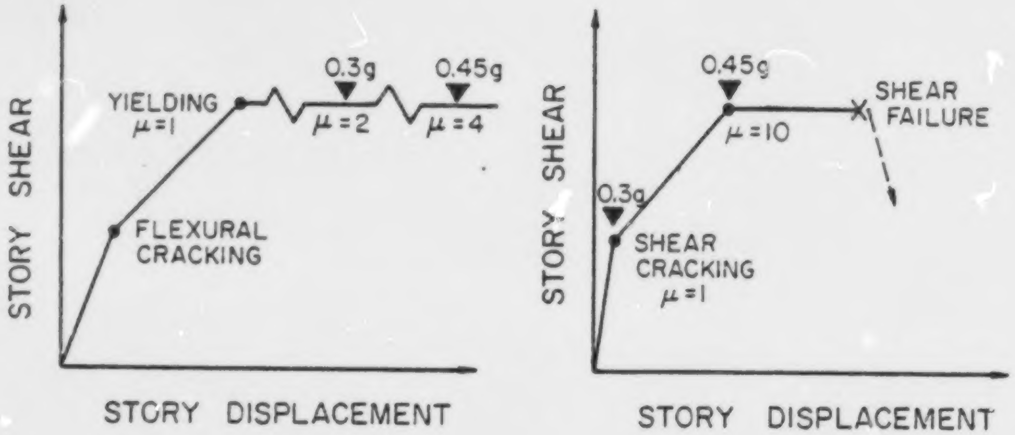
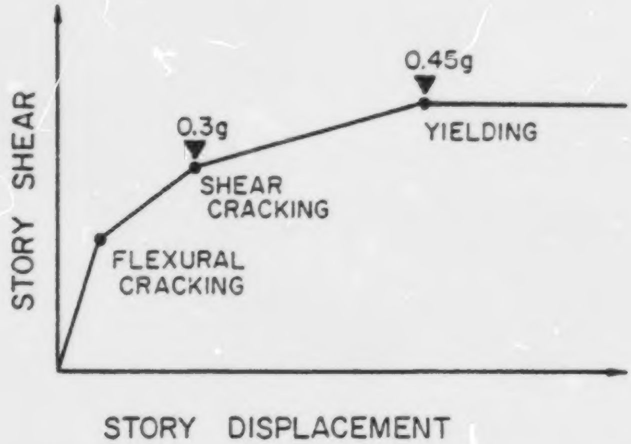

START
STEP (A)
STRUCTURAL MODELING
YES
MODIFICATION OF STRUCTURAL MODELING?
NO
STEP (B)
EVALUATION OF BEHAVIOR UNDER LATERAL FORCES
YES
RE-EVALUATION OF BEHAVIOR UNDER LATERAL FORCES?
NO
LINEAR RESPONSE ANALYSIS NECESSARY?
NO
YES
STEP (C)
LINEAR RESPONSE ANALYSIS
YES
ENOUGH STRENGTH?
UNCERTAIN
STEP (D)
NONLINEAR RESPONSE ANALYSIS
YES
ENOUGH DUCTILITY?
NO OR UNCERTAIN
PRINT "SAFE"
PRINT "UNSAFE" OR "UNCERTAIN"
YES
UNSAFE?
NO
STEP (E)
SYNTHESIS EVALUATION OF SAFETY
SYNTHESIS EVALUATION OF SAFETY
STOP
GO TO NEXT SCREENING

FIGURE 1 FLOW DIAGRAM FOR EVALUATION OF SAFETY OF EXISTING BUILDINGS


STORY SHEAR
YIELDING
$\mu=1$
0.3g
0.45g
$\mu=2$
$\mu=4$
FLEXURAL
CRACKING
STORY DISPLACEMENT
STORY SHEAR
0.45g
SHEAR
FAILURE
$\mu=10$
0.3g
SHEAR
CRACKING
$\mu=1$
STORY DISPLACEMENT

(a) BENDING TYPE
(DEGRADING TRILINEAR)

(b) SHEAR TYPE
(ORIGIN-ORIENTED)


STORY SHEAR
0.45g
0.3g
YIELDING
SHEAR
CRACKING
FLEXURAL
CRACKING
STORY DISPLACEMENT

(c) SHEAR-BENDING TYPE
(MODIFIED DEGRADING TRILINEAR)

FIGURE 2 DECISION CRITERIA AND TYPE OF FAILURE FOR FIRST SCREENING

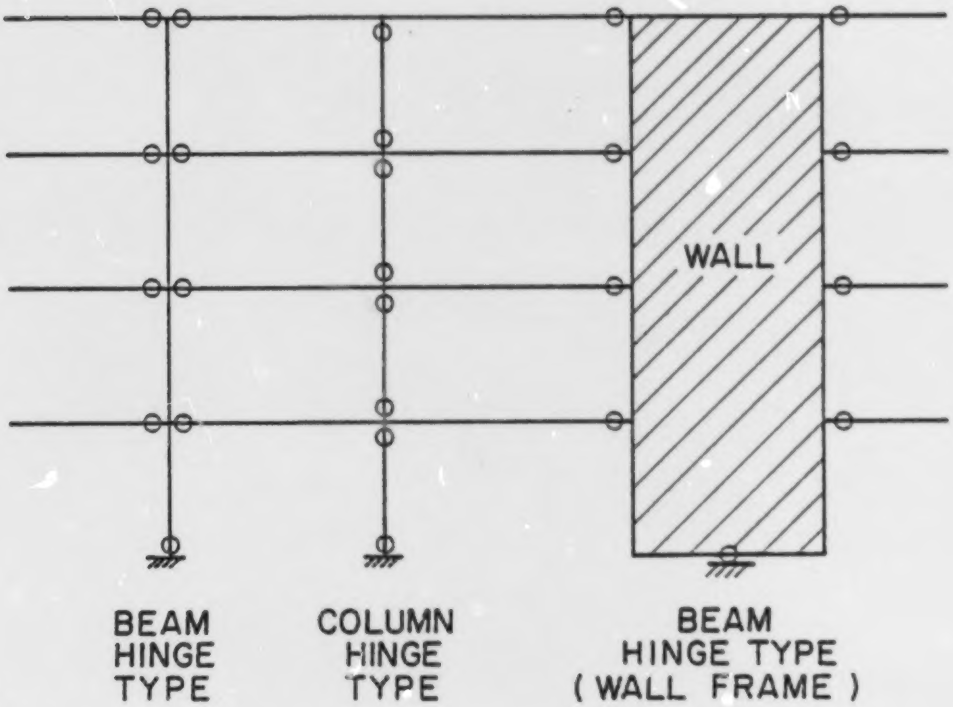
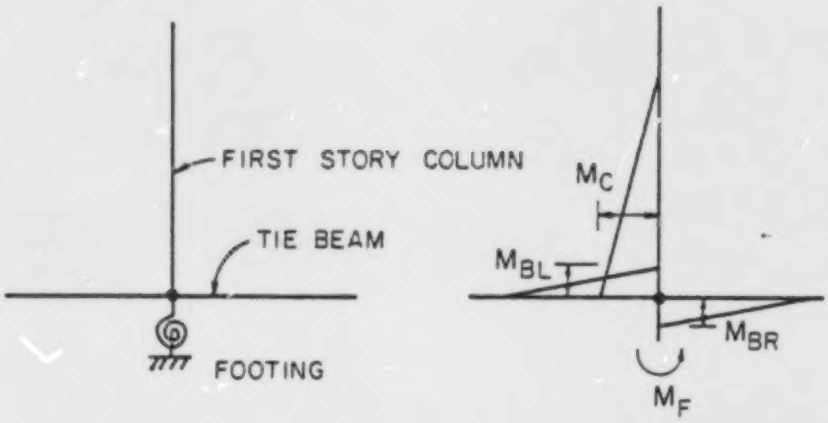
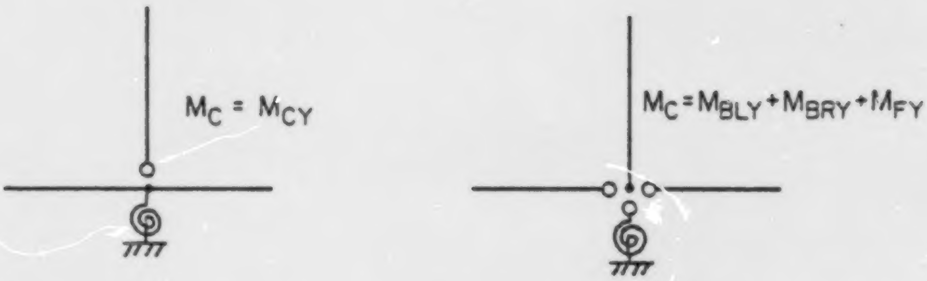

WALL
BEAM HINGE TYPE
COLUMN HINGE TYPE
BEAM HINGE TYPE (WALL FRAME)

FIGURE 3(A) EXAMPLES OF YIELD HINGE MECHANISMS OF FRAME


FIRST STORY COLUMN
TIE BEAM
FOOTING
M_C
M_{BL}
M_{BR}
M_F

$M_C = M_{BR} + M_{BL} + M_F$

STRUCTURAL MODEL EQUILIBRIUM OF MOMENT


$M_C = M_{CY}$
$M_C = M_{BLY} + M_{BRY} + M_{FY}$

$M_{CY} \leq M_{BLY} + M_{BRY} + M_{FY}$ $M_{CY} > M_{BLY} + M_{BRY} + M_{FY}$

COLUMN-HINGE TYPE TIE BEAM-FOOTING TYPE

FIGURE 3(B) YIELD HINGE MECHANISM AT FOOTING LEVEL

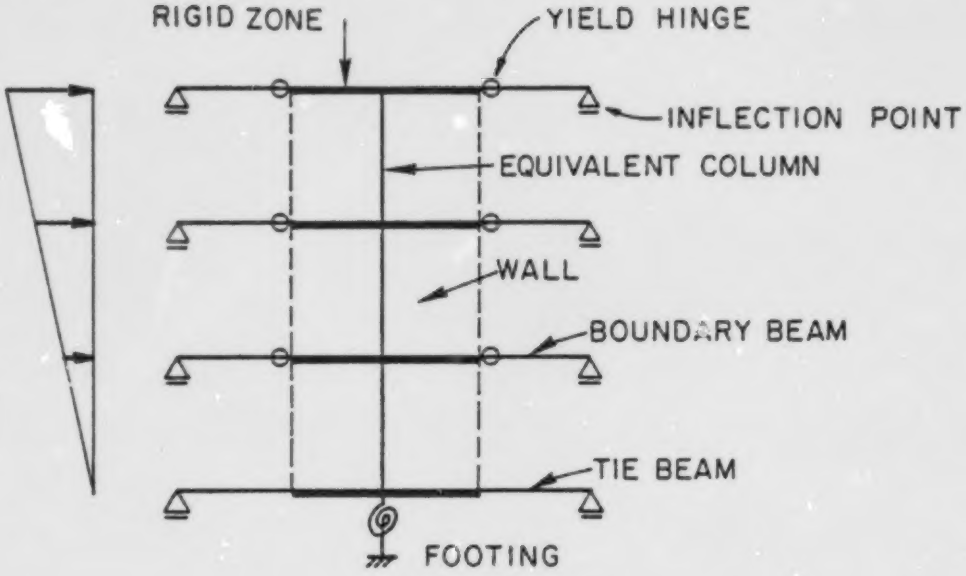
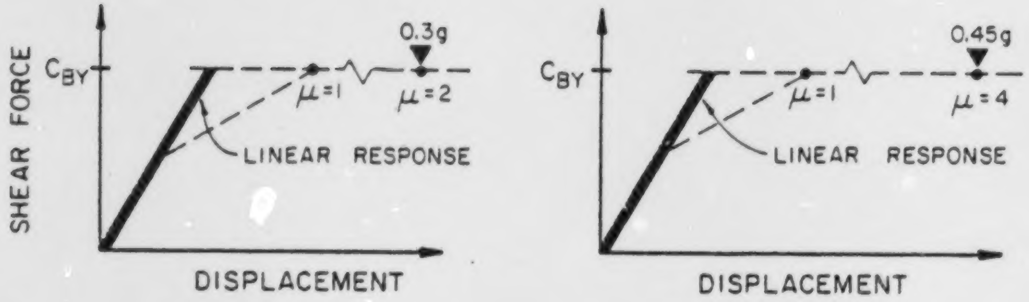

RIGID ZONE
YIELD HINGE
INFLECTION POINT
EQUIVALENT COLUMN
WALL
BOUNDARY BEAM
TIE BEAM
FOOTING

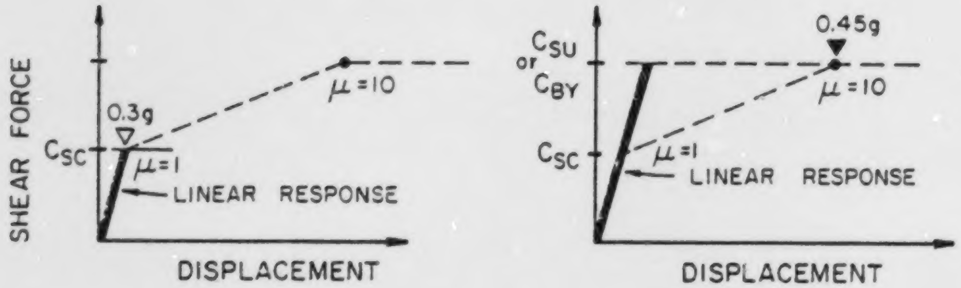
FIGURE 4 EQUIVALENT FRAME FOR SHEAR WALL WITH BOUNDARY BEAMS


SHEAR FORCE
C_BY
0.3g
μ=1
μ=2
LINEAR RESPONSE
DISPLACEMENT
C_BY
0.45g
μ=1
μ=4
LINEAR RESPONSE
DISPLACEMENT

TO 0.3g EARTHQUAKE

TO 0.45g EARTHQUAKE

(a) BENDING TYPE OF BUILDING

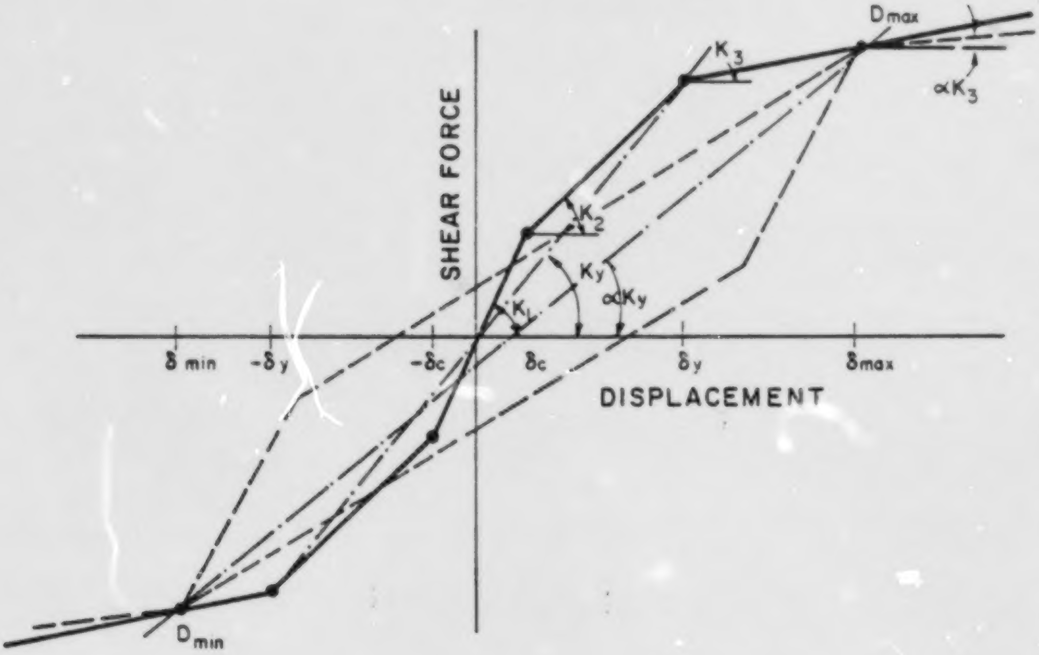

SHEAR FORCE
0.3g
C_SC
μ=1
μ=10
LINEAR RESPONSE
DISPLACEMENT
C_SU
or
C_BY
C_SC
0.45g
μ=10
μ=1
LINEAR RESPONSE
DISPLACEMENT

TO 0.3g EARTHQUAKE

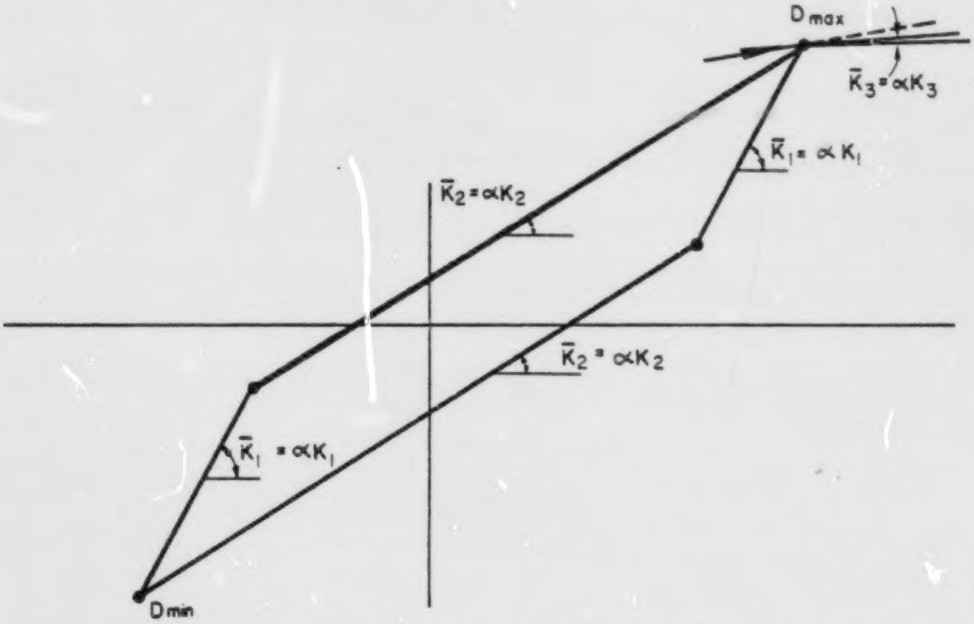
TO 0.45g EARTHQUAKE

(b) SHEAR TYPE OR SHEAR-BENDING TYPE OF BUILDING

FIGURE 5 STRENGTH SAFETY EVALUATION


D_{max}
K_3
αK_3
SHEAR FORCE
K_2
K_y
αK_y
K_1
δ_{min}
$-\delta_y$
$-\delta_c$
δ_c
δ_y
δ_{max}
DISPLACEMENT
D_{min}

a) SKELETON CURVE


D_{max}
$\bar{K}_3 = \alpha K_3$
$\bar{K}_1 = \alpha K_1$
$\bar{K}_2 = \alpha K_2$
$\bar{K}_2 = \alpha K_2$
$\bar{K}_1 = \alpha K_1$
D_{min}

b) HYSTERESIS LOOP

FIGURE 6(A) DEGRADING TRI-LINEAR TYPE

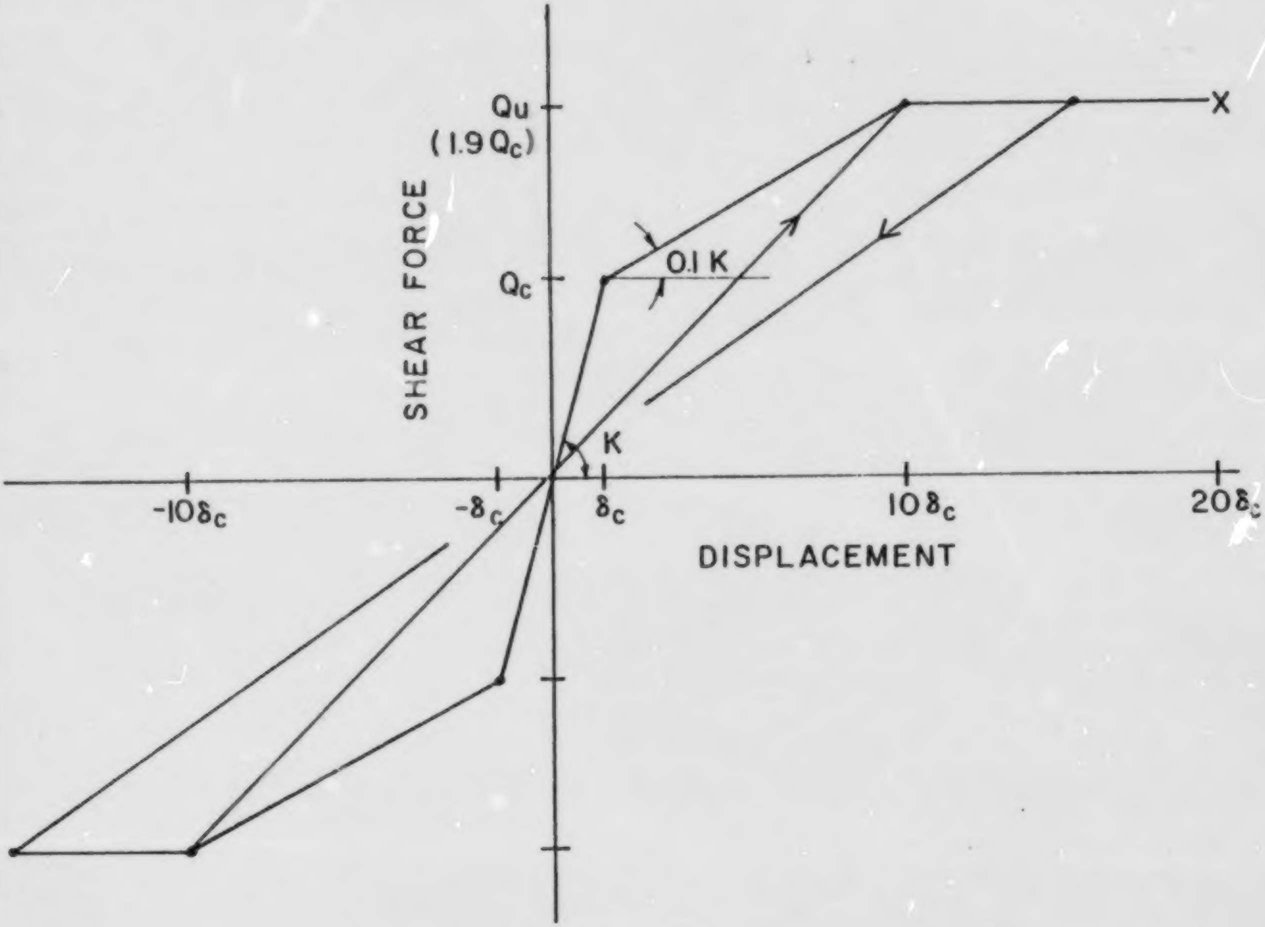
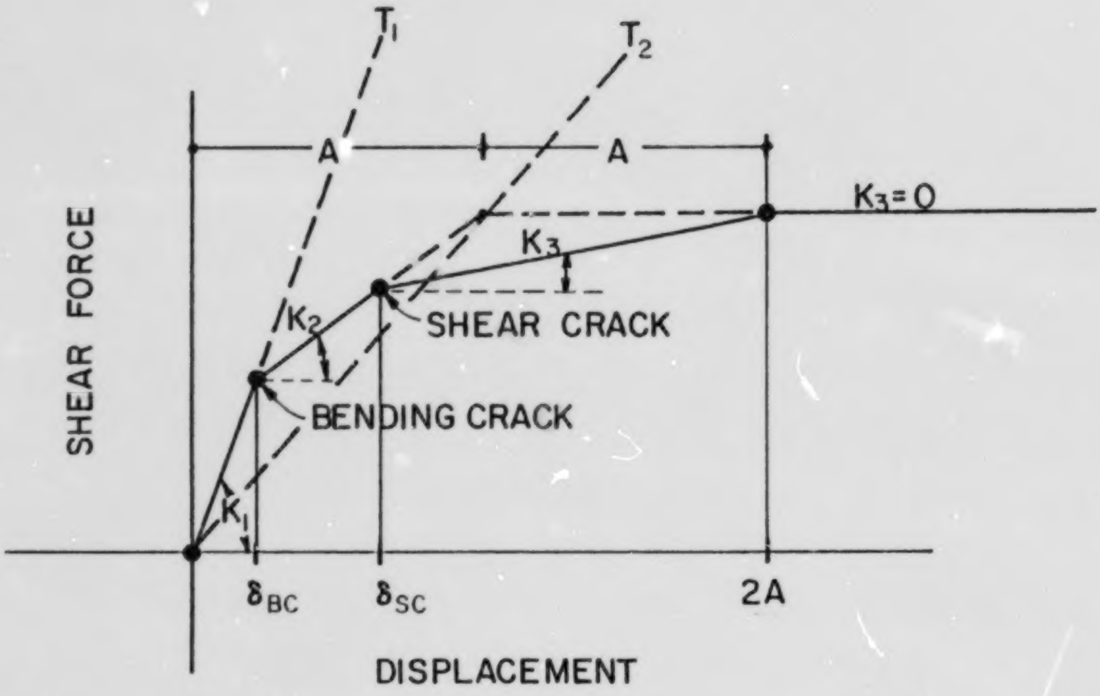
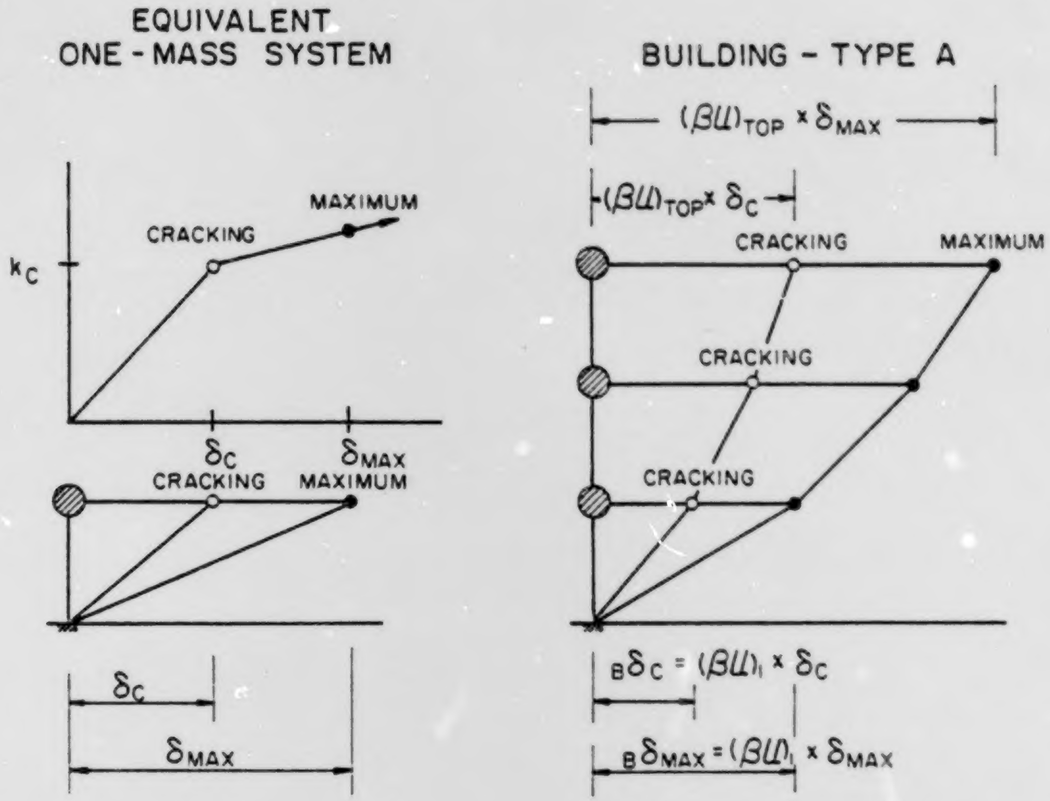

Qu
(1.9 Q_c)
SHEAR FORCE
Q_c
0.1 K
K
X
$-10\delta_c$
$-\delta_c$
δ_c
$10\delta_c$
$20\delta_c$
DISPLACEMENT

FIGURE 6(B) ORIGIN-ORIENTED TYPE


T1
T2
A
A
K3=0
K3
K2
SHEAR CRACK
BENDING CRACK
SHEAR FORCE
K1
δBC
δSC
2A
DISPLACEMENT

DUCTILITY FACTOR (μ) = MAXIMUM DISPLACEMENT / δ_{SC}

FIGURE 6(C) MODIFIED DEGRADING TRI-LINEAR TYPE


EQUIVALENT ONE-MASS SYSTEM
k_C
CRACKING
MAXIMUM
δ_C
δ_MAX
CRACKING
MAXIMUM
δ_C
δ_MAX
BUILDING - TYPE A
(βu)_TOP × δ_MAX
(βu)_TOP × δ_C
CRACKING
MAXIMUM
CRACKING
CRACKING
_Bδ_C = (βu)_1 × δ_C
_Bδ_MAX = (βu)_1 × δ_MAX

ONE-MASS SYSTEM

Cracking Strength = k_c

Cracking Displacement = δ_c

Maximum Displacement = δ_{max}

Maximum Ductility = $\mu_0 = \dfrac{\delta_{max}}{\delta_c}$

Initial Natural Period = T_1

FIRST STORY OF BUILDING TYPE A

$$_BC_c = k_c \quad \frac{\sum_{i=1}^{n} \overline{W}_i}{\sum_{i=1}^{n} (\beta u)_i \overline{W}_i}$$

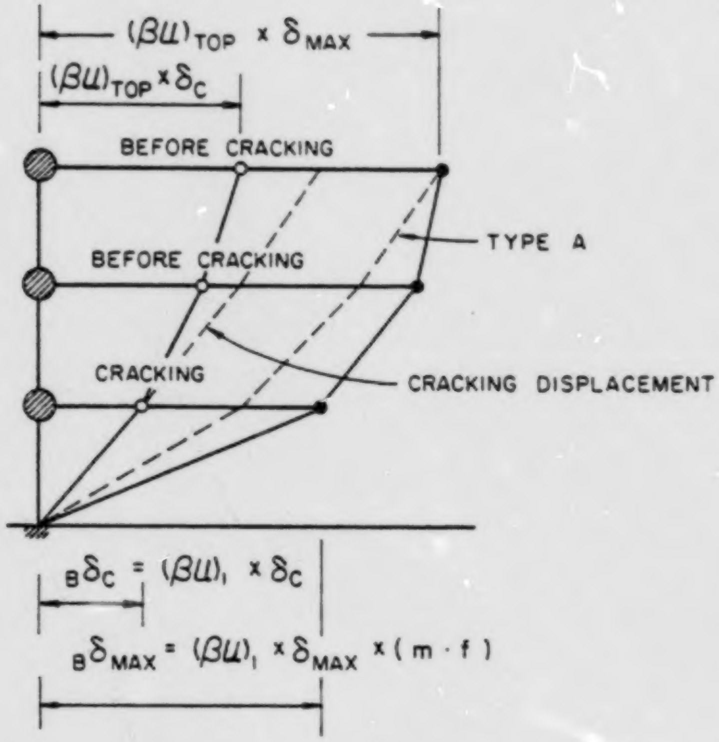
$$_B\delta_c = (\beta u)_1 \times \delta_c$$

$$_B\delta_{max} = (\beta u)_1 \times \delta_{max}$$

$$_B\mu = \mu_0 \qquad _BT_1 = T_1$$

FIGURE 7 CONVERSION OF STRENGTH AND DISPLACEMENT

BUILDING - TYPE B


$(\beta u)_{TOP} \times \delta_{MAX}$
$(\beta u)_{TOP} \times \delta_{C}$
BEFORE CRACKING
BEFORE CRACKING
TYPE A
CRACKING
CRACKING DISPLACEMENT
${}_B\delta_C = (\beta u)_1 \times \delta_C$
${}_B\delta_{MAX} = (\beta u)_1 \times \delta_{MAX} \times (m \cdot f)$

FIRST STORY OF BUILDING TYPE B

$${}_B C_c = k_c \frac{\sum_{i=1}^{n} \overline{W}_i}{\sum_{i=1}^{n} (\beta u)_i \overline{W}_i}$$

$${}_B\delta_c = (\beta u)_1 \times \delta_c$$

$${}_B\delta_{max} = (\beta u)_1 \times \delta_{max} \times (m \cdot f)$$

$${}_B\mu_o = \mu_o \times (m \cdot f)$$

$${}_B T_1 = T_1$$

FIGURE 7 CONVERSION OF STRENGTH AND DISPLACEMENT (CONT.)

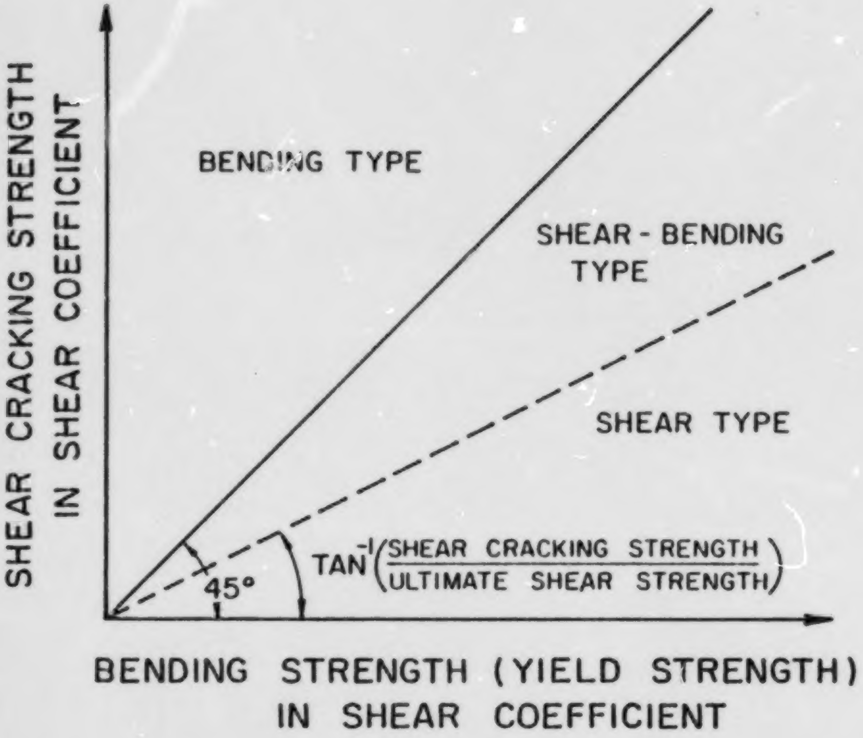
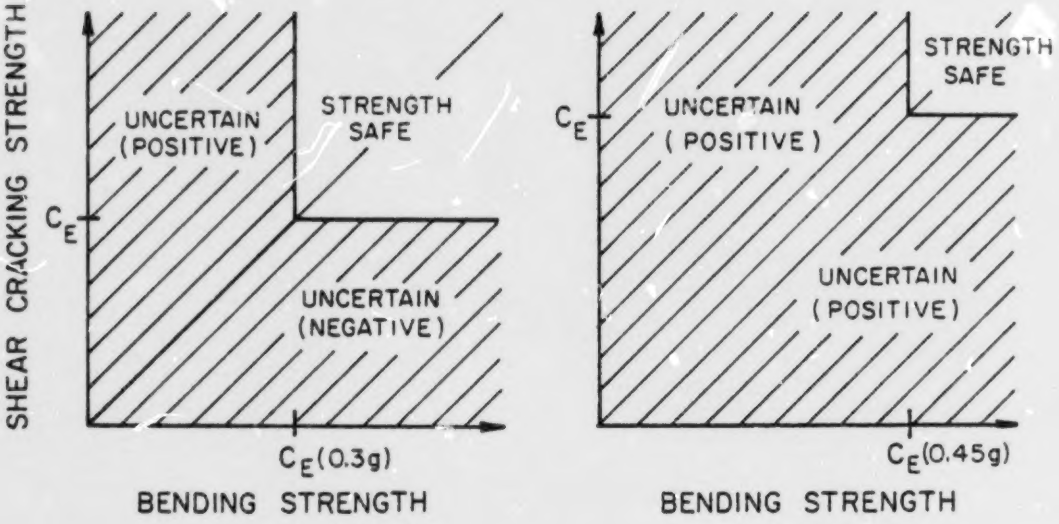

SHEAR CRACKING STRENGTH IN SHEAR COEFFICIENT
BENDING TYPE
SHEAR - BENDING TYPE
SHEAR TYPE
45°
TAN^{-1}(SHEAR CRACKING STRENGTH / ULTIMATE SHEAR STRENGTH)
BENDING STRENGTH (YIELD STRENGTH) IN SHEAR COEFFICIENT

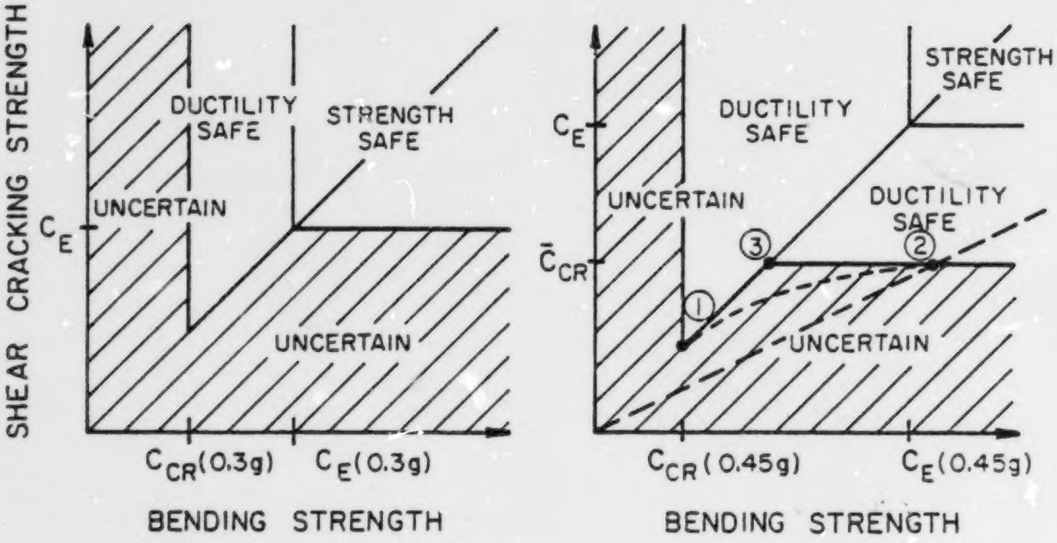
FIGURE 8 CLASSIFICATION OF TYPE OF FAILURE - STEP B


SHEAR CRACKING STRENGTH
UNCERTAIN (POSITIVE)
STRENGTH SAFE
C_E
UNCERTAIN (NEGATIVE)
$C_E(0.3g)$
BENDING STRENGTH
C_E
UNCERTAIN (POSITIVE)
STRENGTH SAFE
UNCERTAIN (POSITIVE)
$C_E(0.45g)$
BENDING STRENGTH

0.3g EARTHQUAKE

0.45g EARTHQUAKE

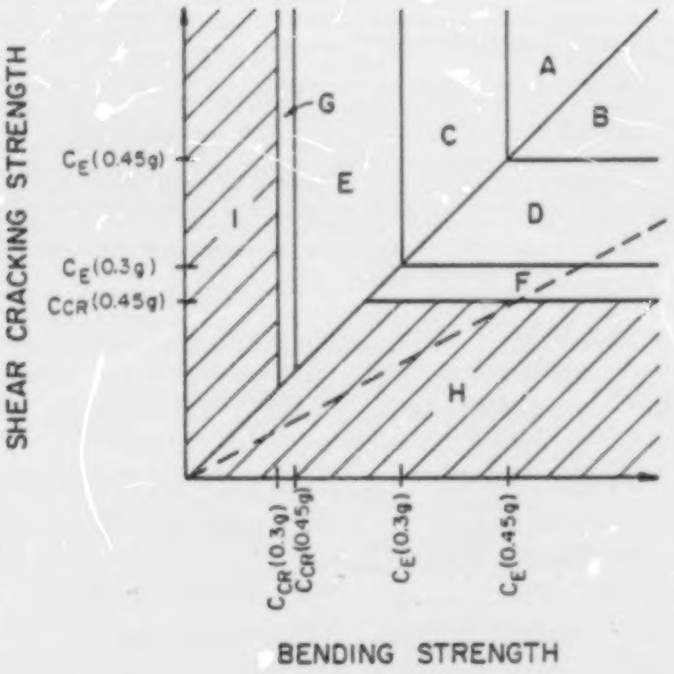
FIGURE 9 STRENGTH SAFETY EVALUATION - STEP C


SHEAR CRACKING STRENGTH
DUCTILITY SAFE
STRENGTH SAFE
UNCERTAIN
C_E
UNCERTAIN
$C_{CR}(0.3g)$
$C_E(0.3g)$
BENDING STRENGTH
STRENGTH SAFE
C_E
DUCTILITY SAFE
UNCERTAIN
DUCTILITY SAFE
③
②
$\bar{C}_{CR}$
①
UNCERTAIN
$C_{CR}(0.45g)$
$C_E(0.45g)$
BENDING STRENGTH

0.3g EARTHQUAKE

0.45g EARTHQUAKE

FIGURE 10 DUCTILITY SAFETY EVALUATION - STEP D

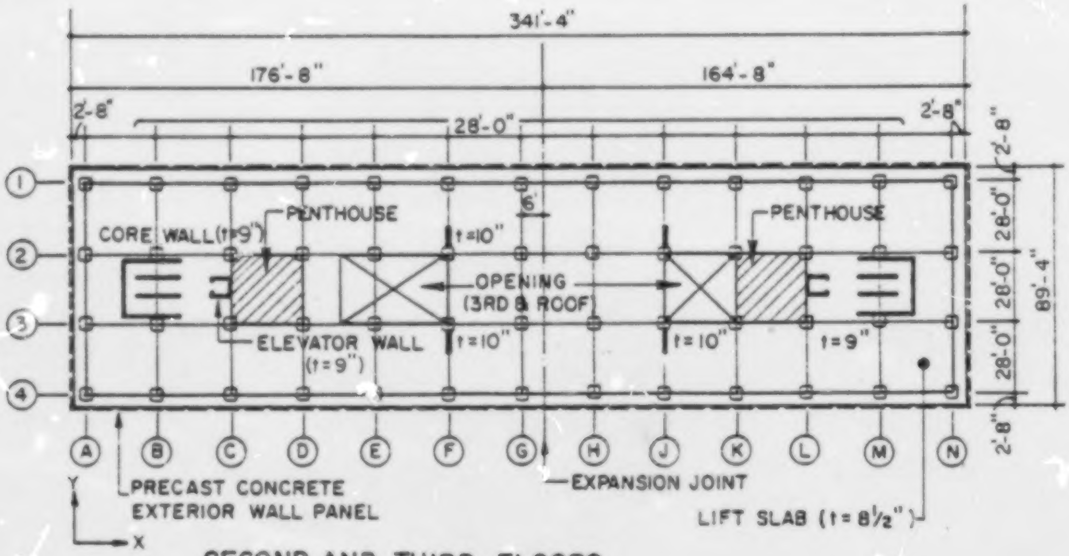

SHEAR CRACKING STRENGTH
A
B
C
D
E
F
G
H
I
C_E(0.45g)
C_E(0.3g)
C_CR(0.45g)
C_CR(0.3g)
C_CR(0.45g)
C_E(0.3g)
C_E(0.45g)
BENDING STRENGTH

(A) SHEAR CRACKING STRENGTH - BENDING STRENGTH DIAGRAM

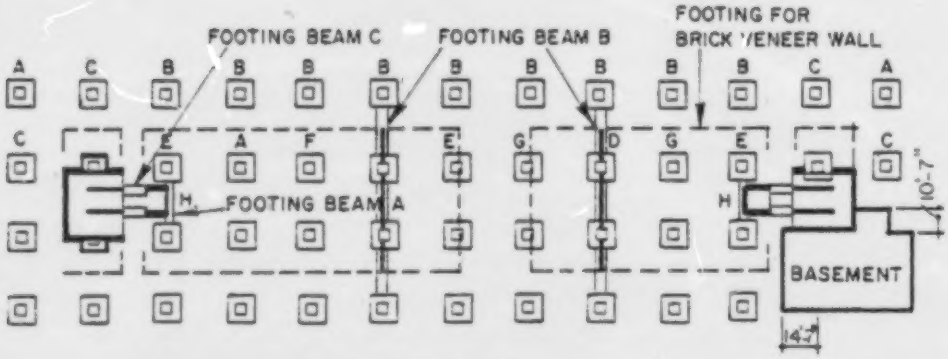
		0.3g Earthquake		0.45g Earthquake		
Zone	Type	Strength	Ductility	Strength	Ductility	Ranking
A	Bending	Safe	Safe	Safe	Safe	I
B	Shear or Shear-Bending	Safe	Safe	Safe	Safe	
C	Bending	Safe	Safe	Uncertain	Safe	
D	Shear or Shear-Bending	Safe	Safe	Uncertain	Safe	
E	Bending	Uncertain	Safe	--	Safe	II
F	Shear or Shear-Bending	Uncertain	Uncertain	--	Safe	III
G	Bending	--	Safe	--	Uncertain	
H	Shear or Shear-Bending	--	--	--	Uncertain	IV
I	Bending	--	--	--	Uncertain	

(B) RANKING OF SAFETY

FIGURE 11 SYNTHESIS EVALUATION OF SAFETY - STEP E

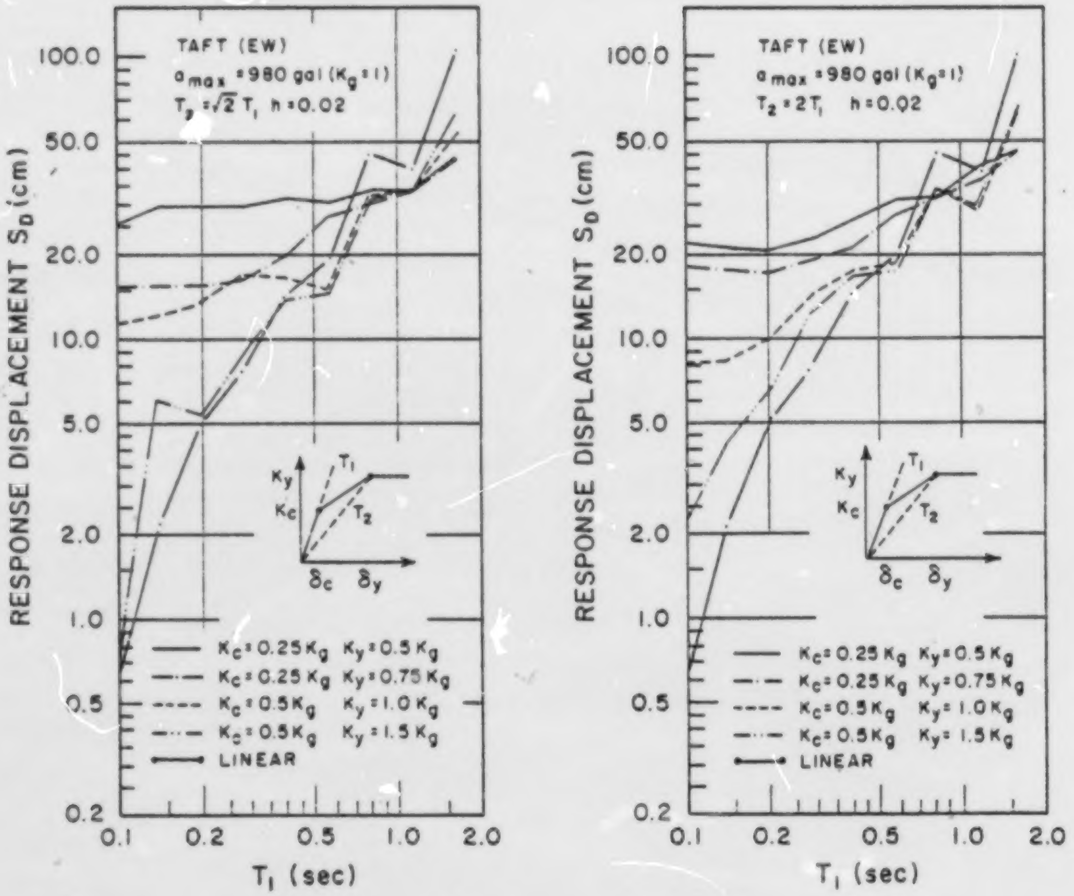

341'-4"
176'-8"
164'-8"
2'-8"
28'-0"
2'-8"
2'-8"
28'-0"
28'-0"
28'-0"
89'-4"
2'-8"
1
2
3
4
CORE WALL (t=9")
PENTHOUSE
t=10"
6'
PENTHOUSE
OPENING
(3RD & ROOF)
ELEVATOR WALL
(t=9")
t=10"
t=10"
t=9"
A
B
C
D
E
F
G
H
J
K
L
M
N
Y
X
PRECAST CONCRETE
EXTERIOR WALL PANEL
EXPANSION JOINT
LIFT SLAB (t=8½")

SECOND AND THIRD FLOORS


FOOTING BEAM C
FOOTING BEAM B
FOOTING FOR
BRICK VENEER WALL
A
C
B
B
B
B
B
B
B
B
B
C
A
C
E
A
F
E
G
D
G
E
C
H
FOOTING BEAM A
H
10'-7"
BASEMENT
14'-7"

FOOTING

FIGURE 12 PLAN OF SCHOOL BUILDING A



(a)

(b)

FIGURE 13 RESPONSE SPECTRA FOR DEGRADING TRI-LINEAR TYPE (AFTER REFERENCE 2)

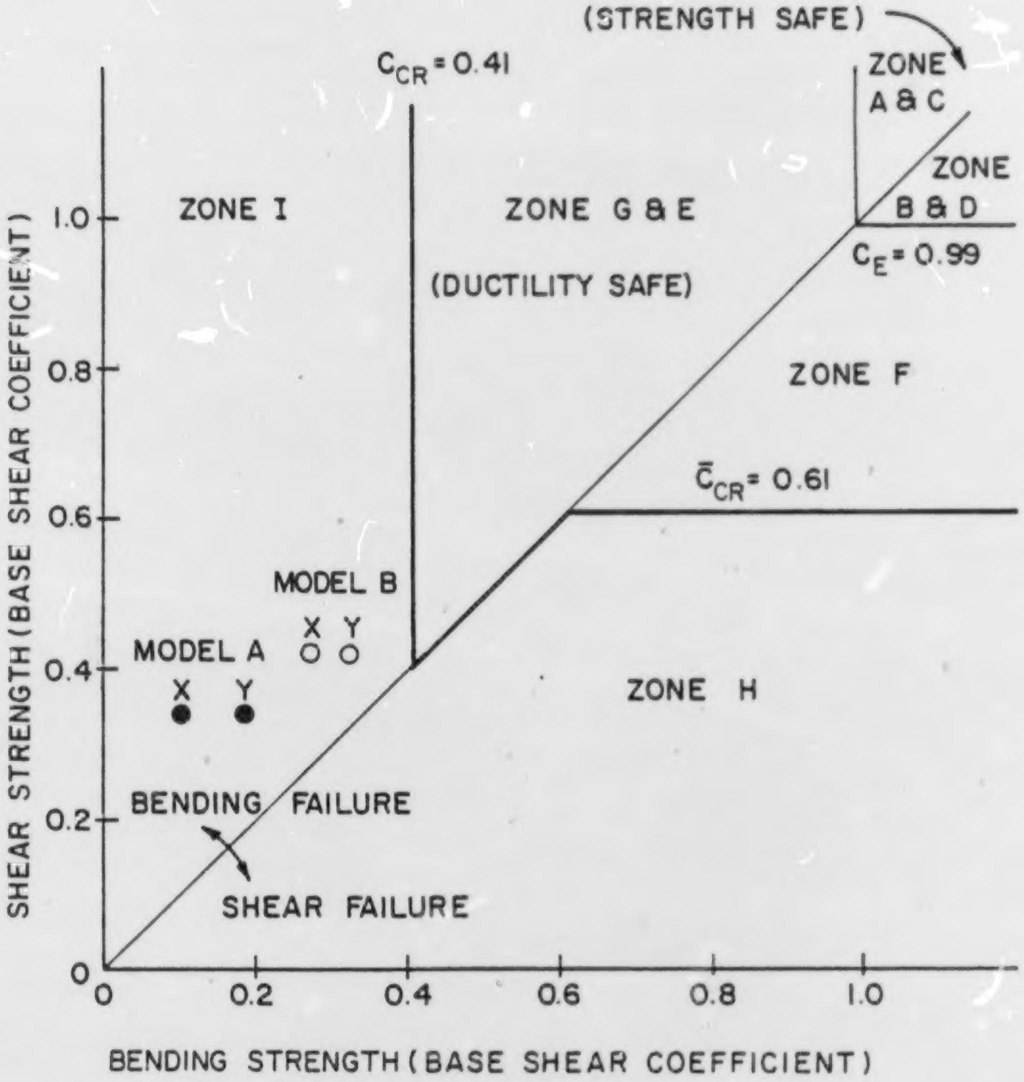
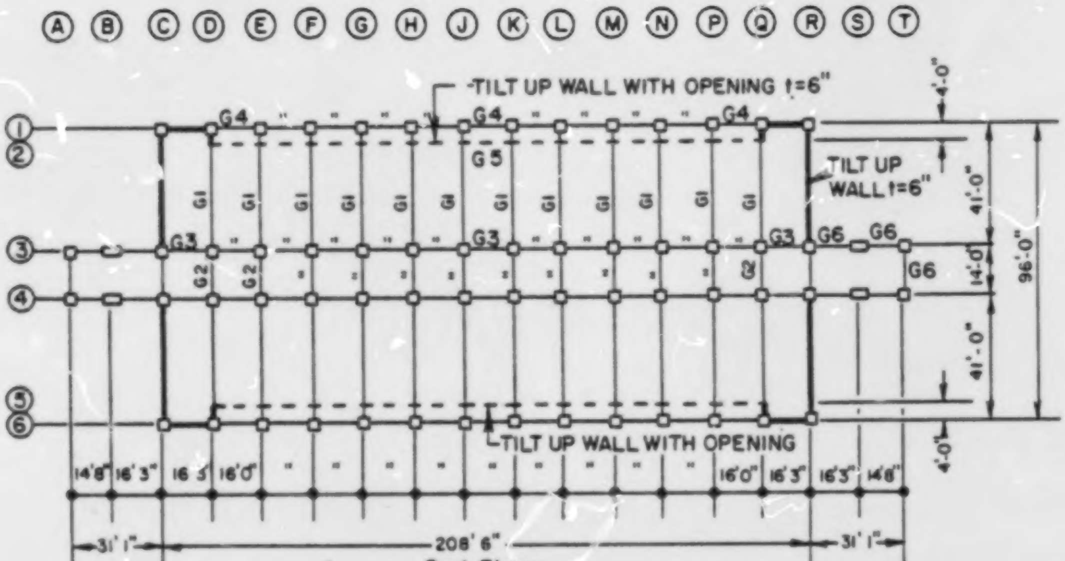
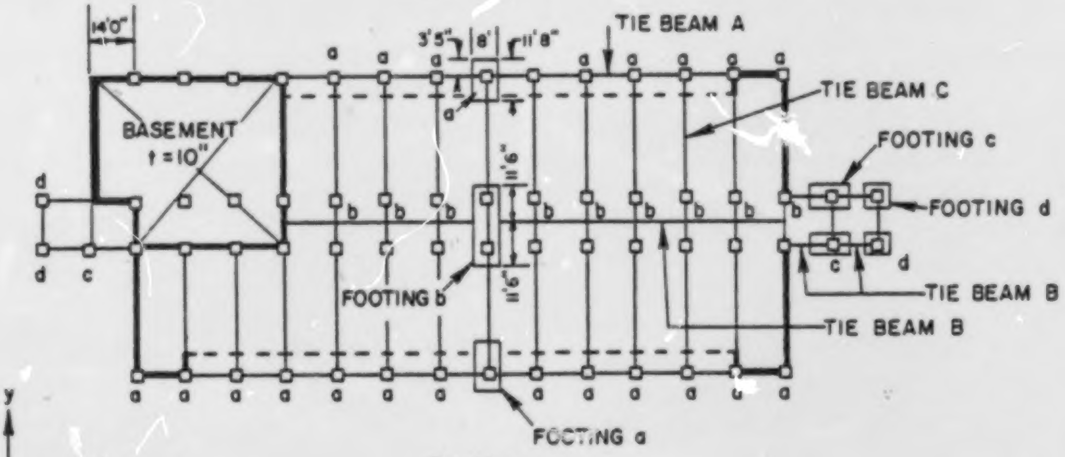


FIGURE 14 SYNTHESIS EVALUATION OF SAFETY OF SCHOOL BUILDING A


A B C D E F G H J K L M N P Q R S T
1
2
3
4
5
6
TILT UP WALL WITH OPENING t=6"
G4
G4
G4
G5
TILT UP WALL t=6"
G1
G3
G3
G3
G6
G6
G6
G2
4'-0"
41'-0"
14'-0"
96'-0"
41'-0"
4'-0"
TILT UP WALL WITH OPENING
14'8"
16'3"
16'3"
16'0"
16'0"
16'3"
16'3"
14'8"
31'1"
208'6"
31'1"

2nd Floor


14'0"
3'5"
8'
11'8"
TIE BEAM A
TIE BEAM C
BASEMENT t=10"
FOOTING c
FOOTING d
11'6"
11'6"
FOOTING b
TIE BEAM B
TIE BEAM B
FOOTING a
a
b
c
d
y
x

Footing

FOOTING a: 8'0" x 11'8"

" b: 8'0" x 23'0"

" c: 5'0" x 12'0"

" d: 5'0" x 5'0"

TIE BEAM A: B x D = 8" x 2'0"

TOP BARS: 2- #4
BOTTOM : 2- #4

STIRRUP : #4 @ 12"

TIE BEAM B & C: B x D = 12" x 12"

TOP : 2- #5
BOTTOM: 2- #5
STIRRUP: #3 @ 12"

FIGURE 15 PLAN OF SCHOOL BUILDING B

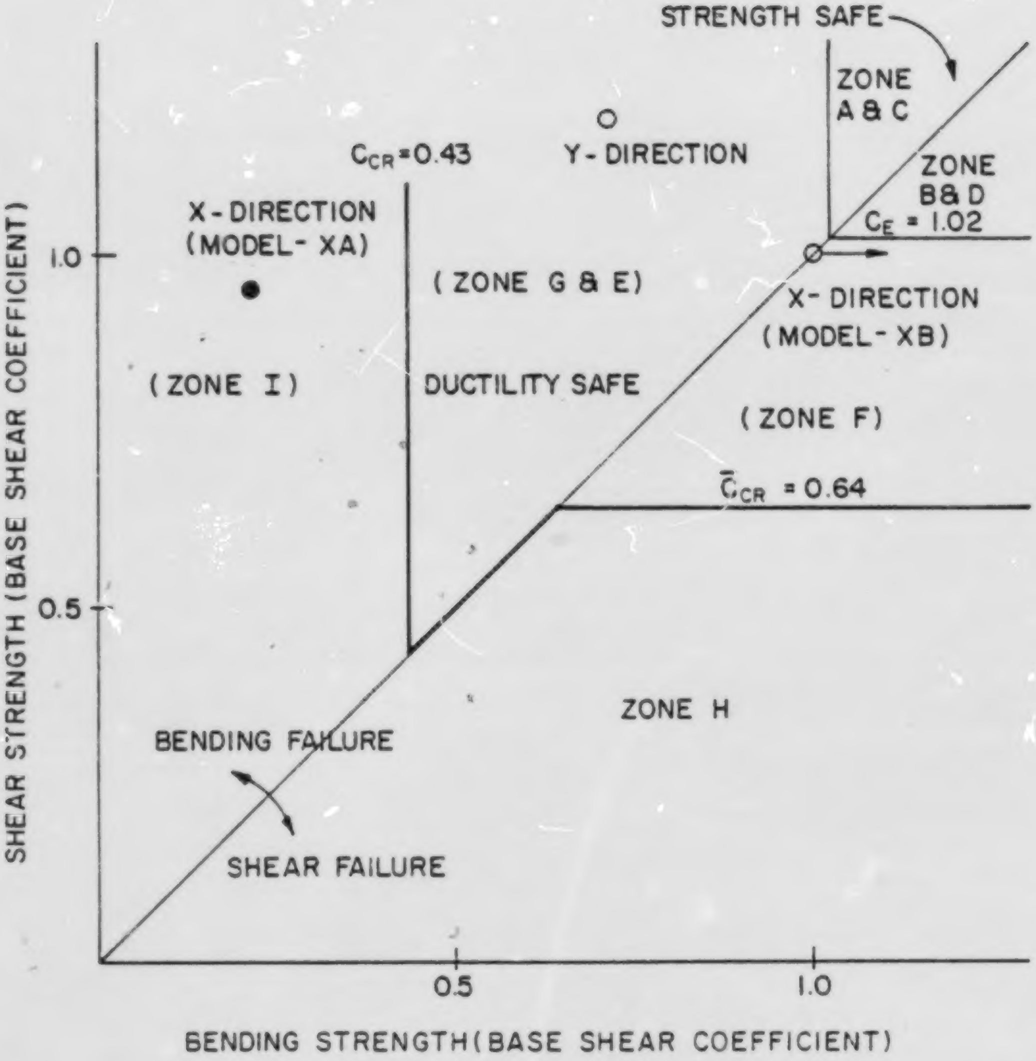
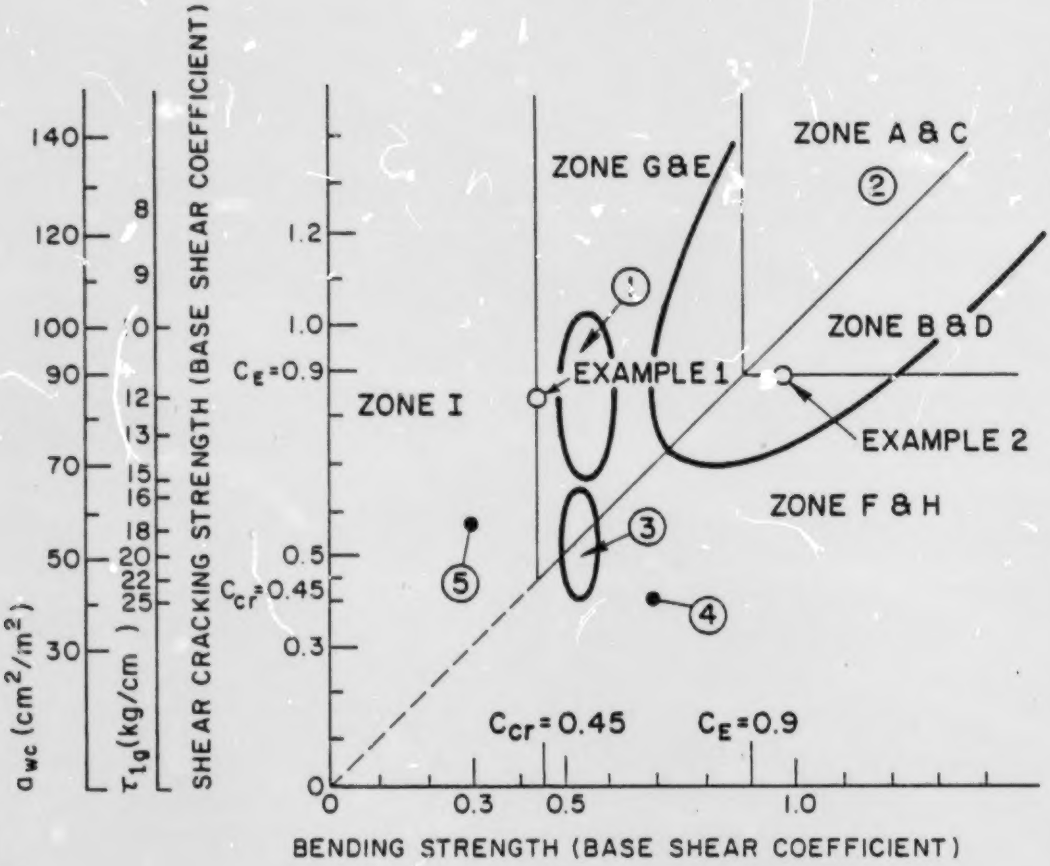


FIGURE 16 SYNTHESIS EVALUATION OF SAFETY OF SCHOOL BUILDING B



① SCHOOL BUILDINGS OF TYPE C, NO DAMAGE
② MANY SHEAR WALLS, NO DAMAGE
③ MEDIUM DAMAGE
④ HACHINOHE TECHNICAL HIGH SCHOOL, SEVERE DAMAGE
⑤ HACHINOHE LIBRARY, SEVERE DAMAGE

EXAMPLES 1 & 2 ARE DISCUSSED IN REFERENCE 1.

FIGURE 17 CHARACTERISTICS OF BUILDINGS IN 1968 TOKACHI-OKI EARTHQUAKE (AFTER REF. 3)

DESIGN AND ENGINEERING DECISIONS:
FAILURE CRITERIA (LIMIT STATES)

by

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PANEL ON DESIGN AND ENGINEERING DECISIONS: FAILURE CRITERIA (LIMIT STATES)

by V. Bertero[1] and B. Bresler[1]

INTRODUCTION

Aseismic design is only one aspect of the design process. In this process, the designer must establish functional and environmental demand conditions on a building and acceptable levels of performance under these conditions. In terms of aseismic design, this requirement calls for establishing critical design earthquake or earthquakes and corresponding acceptable levels of performance or failure criteria. Usually, this problem is stated in terms of establishing design loads and their critical combinations and in terms of permissible limits of structural response under these loading conditions.

The establishment of appropriate loadings and their critical combinations requires decisions as to failure criteria and is the most difficult problem in the design process. One of the major difficulties in establishing such loadings and combinations is the uncertainty associated with predicting future ground motions and that associated with the complex behavior of soil-building systems under severe ground motions. An additional problem is caused by socio-economic requirements for greatest safety at a least reasonable cost. In order to optimize a design or to maximize utility [1], an estimate of economic losses resulting from failure is required. The term failure as used herein is synonymous with "inadmissible limit states" and includes all modes of undesirable behavior, from damage to cosmetic appearance to collapse, which may render buildings unfit for use [1].

OBJECTIVES AND SCOPE. - Other contributions to the Panel on Design and Engineering Decisions will deal with problems of optimization, consequences of failure, and codes. Therefore, the main objective of this paper is to discuss the failure criteria (inadmissible limit states) which should be considered in aseismic design of buildings. After discussing the principal failure criteria (serviceability and ultimate limit states) presently used in design, results from surveys and analyses of building damage during recent earthquakes are briefly reviewed. These recent observations indicate that an additional category of limit states related to damage which cannot be properly assigned to either serviceability failure or inadmissible ultimate limit states is needed. A discussion of damageability criteria and possible forms of damageability indices is included. Observations of damage in recent earthquakes have clearly indicated that a significant number of existing buildings are hazardous and may suffer varying degrees of damage even under moderate earthquakes. The cumulative effects of aging and other sources of possible distress--such as extreme climatic environment, wind, and fire--must therefore be considered in designing new buildings and in evaluating hazards in existing buildings.

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DESIGN BASED ON LIMIT STATES

DEFINITION OF LIMIT STATES. - All structures must be designed to sustain safely all loads and deformations liable to occur during construction and in use, and to have adequate durability during its service life. A structure, or a part of a structure, is rendered unfit for use when it reaches a particular state, called a "limit state," in which it ceases to fulfill the function or to satisfy the conditions for which it was designed [2]. To define the different limit states, it is necessary to identify the various events that might lead to some cost of "disutility" to the occupant, owner, or designer. The different limit states are presently grouped as either serviceability or ultimate limit states. The events normally considered in limit state design and the applications of limit state philosophy to practical design methods are discussed in Refs. 2, 3, and 4. The format used in formulating the limit state design philosophy encourages the use of probabilistic methods where sufficient statistical information is available [3,4]. Because of uncertainties involved in defining the design earthquake, as well as the structural parameters controlling the mechanical behavior of a building, a probabilistically formulated limit state design philosophy is well-suited for developing aseismic design methods. A logical approach to the aseismic design of a structure is that of comprehensive design.

COMPREHENSIVE DESIGN. - Sawyer [5] discussed a comprehensive design procedure in which the resistance of the structure to the various failure stages is correlated with the probability of the corresponding excitations, so that the total cost, including the first cost and the expected losses from all the limit stages, is minimized. Failure of a structure under increasing loads generally occurs in successively more severe stages under successively less probable levels of load. To illustrate this point, the relationship shown in Fig. 1 shows the failure stages versus a monotonically increasing pseudo-static load for a typical statically indeterminate reinforced concrete building. Due to the variability of loss for a given load (or the variability of load for a given loss), the relationship shown in Fig. 1 should be considered as representing mean values of the random variables involved. The full redistribution, as shown in Fig. 2, can, in some cases, involve large variances [6].

In comprehensive design, identification of the potential modes of failure requires prediction of the mechanical behavior of a structure at each significant level of critical combinations of all possible excitations to which the structure may be subjected. Because it is usually not possible to consider real behavior under the actual critical excitations to which the structure may be subjected, it is common to base structural design on idealized conceptions of mechanical behavior under a simplified set of excitations. The sources, treatment, and effects of the different types of excitations which may be exerted on structures are summarized in Fig. 3 [7]. The sequence of actions to which a structure may be subjected often consists of unpredictable fluctuations in the magnitude, direction and/or position of each of the individual excitations. The only characteristics that may be estimated accurately are the extreme values between which each of these actions will oscillate. These types of actions have been classified in Fig. 3 as generalized or variable-repeated excitations.

The particular phenomena associated with variable-repeated excitations are classified as long-endurance fatigue, low-cycle fatigue, and incremental collapse. Long-endurance fatigue is a critical consideration only in special structures. A review of results regarding low-cycle fatigue, which is associated with repeated-reversible actions, indicates that the real danger of these actions is not fracture of the structural material, but deterioration of the stiffness, particularly in the case of reinforced concrete [7]. Incremental collapse is associated with progressive development of excessive deflections which occur under the cyclic applications of different combinations of peak actions. Because deterioration of stiffness can lead to an undesirable increase in deformations, in examining actual generalized excitations, the effects of alternating excitations cannot be treated independently, as is usually done, from those caused by excitation patterns leading to incremental deformations [7].

CURRENT FAILURE CRITERIA IN ASEISMIC DESIGN

GENERAL GOALS AND CURRENT PRACTICE. - The general philosophy of earthquake resistant design for buildings other than essential facilities has been well-established and proposed to: (1) prevent nonstructural damage in minor earthquake ground shakings which may frequently occur in the service life of the structure, (2) prevent structural damage and minimize nonstructural damage in moderate earthquake shakings which may occasionally occur, and (3) avoid collapse or serious damage in major earthquake ground shakings which may rarely occur. This philosophy is in complete accordance with the concept of comprehensive design. Current design methodologies, however, fall short of realizing the objectives of this general philosophy. Application of the comprehensive design approach to aseismic design would entail replacing the load and load probability scales by the seismic excitation intensity and intensity probability scales, respectively (Figs. 1 and 2). Practical application of this approach is, however, considerably more complex because of difficulties involved in assessing the relationship between loss and seismic excitation. According to the concept of comprehensive design, the ideal design is that which results in the minimum total cost, including possible losses, for all limit states. However, this ideal is not an immediate practical possibility in actual design. No practical design method has yet been developed that satisfies simultaneously all the requirements imposed by the different limit states. In practice, the most critical limit state is used as the basis for proportioning members in the preliminary design; all other main limit states should then be checked through a comprehensive analysis. The advantages of developing a design method based on two failure stages have been discussed by Sawyer [5], and a design method based on two behavior criteria (collapse and loss of serviceability) and on four optimizing criteria has been developed [8]. Application of this method to the aseismic design of ductile moment-resisting frames seems feasible and practical [9].

Because current design practice in regions of high seismic risk focusses on collapse of the main structure as the controlling limit state, the resulting design must be checked for serviceability requirements under normal loading conditions. Examination of building damage resulting from recent severe seismic ground shaking reveals that although buildings were far from reaching the collapse limit state, the degree of nonstructural damage was so great as to constitute failure. Therefore, it is desirable to introduce a new group of limit states based on damageability. Before discussing this need in more detail, the failure criteria used in present aseismic design practice should be considered.

SERVICEABILITY REQUIREMENTS. - Although the conditions leading to serviceability limit states under normal loading have been defined in general terms [2], specific quantitative limits have not been adequately determined. More practical and consistent quantifications are needed for determining failure stages of structural and nonstructural components under all types of service excitations. For example, it has been recommended that the maximum tolerable drift index for walls be limited to 0.002 [10]. On the other hand, in the case of seismic loads, the 1976 Uniform Building Code (UBC) specifies a maximum index of 0.005. Since seismic forces specified in this code apply to designs at service load levels, the UBC value for seismic drift appears to be unconservative when compared to that suggested in Ref. 10.

In quantifying the serviceability limit states for seismic excitations, it is necessary to determine the building's function and the level of excitation intensity under which the facility should remain serviceable. In the case of essential facilities, these should not only be safe, but they should be functional for emergency purposes even after the occurrence of the maximum credible excitations expected during the service life of the building. Some quantitative limits for serviceability requirements for essential facilities are shown in Table 1. Although the seismic design forces for the different codes considered in this table are not strictly comparable, the significant differences between these specified tolerable drift indices indicate the need for more thoroughly investigating the degree of damage constituting failure and corresponding tolerable drift criteria.

ULTIMATE OR SAFETY REQUIREMENTS. - Analysis of the causes leading to ultimate failure of the building reveals that this can be induced by different failure mechanisms acting independently or in combination. Some of these limit states appear to be extremely critical under pseudo-static loads, while they may be negligible under dynamic loads. Under a sustained pseudo-static overload, for example, the limit state caused by transformation of the structure into a mechanism leads to instability of the whole structure; this is usually not so under dynamic loading. Actually, present aseismic design methods are based on the assumption that large displacements (large ductility) develop after the structure is transformed into a mechanism. The distinction between pseudo-static and dynamic effects also applies in the case of ultimate limit state caused by deformation instability.

Failures under Generalized Dynamic Excitations. - Collapse of a structure can occur as a consequence of "low-cycle fatigue" or "incremental deformations" under excitation intensities lower than those required to induce instantaneous collapse if these excitations are considered as monotonically increasing. As pointed out in Refs. 1 and 7, cumulative damage resulting from a long, strong ground motion, a short main shock followed by a succession of aftershocks, or a combination of the main shock and another consequential event or environmental exposure such as fire, can lead to either one of the above two phenomena and therefore merits considerably more attention that it has received.

Yamada and Kawamura [11] have discussed an ultimate aseismic design philosophy of reinforced concrete based on low-cycle fatigue. This type of failure is very sensitive to detailing and quality control of materials and workmanship used in construction. If errors in design or construction, or lack of quality control of materials and of workmanship are eliminated, then application of adequate seismic design provisions with possible further improvements [12], will result in structural designs in which low-cycle fatigue would not control the design. By detailing the expected critical regions of different structural members according to recently proposed seismic code provisions, the energy absorption and energy dissipation capacity developed under cyclic reversals of deformation will be so large as to resist the energy input of even the toughest of credible seismic motions. Even under the most severe ground motions recorded, the number of reversals that can occur between opposite peak deformations having the maximum intensity is not usually large enough to be of serious concern [12]. It should also be noted that under full reversals of symmetrically yielding and strain-hardening or strain-softening structures, the P-Δ effect is cancelled out (Fig. 4).

Studies carried out at Berkeley [13] have shown that one case where low-cycle fatigue could control the design involves members that are used as structural dampers to dissipate energy. One typical example of such a case is that involving coupling girders in coupled wall systems [13]. However, failure of these members does not necessarily lead to complete structural failure. Since these elements act as safety fuses between two different structural resistant systems, their failure would lead to a change in the dynamic characteristics of the system rather than to a brittle failure of the complete system.

A schematic illustration of the incremental collapse, denoted as "crawling collapse," is shown in Fig. 5. Recent studies [14] have shown that this type of failure can control the aseismic design of structure, particularly at sites near the source of seismic ground motions containing severe, long acceleration pulses. For example, the study of the response of a multistory steel frame, optimally designed using a nonlinear method, to seismic ground motions derived from those recorded during the 1971 San Fernando earthquake shows that the frame will collapse due to the type of incremental deformations illustrated by the first story displacement time-history response of Fig. 6. The danger of incremental collapse is aggravated by the high probability that several aftershocks of intensities and dynamic characteristics comparable to that of the main shock will occur. As Newmark and Rosenblueth [1] have pointed out, it is not unusual for a structure which is able to withstand a major shock with visible damage, to collapse during an aftershock.

Although the P-Δ effect is not a factor in failures due to low-cycle fatigue, it is of paramount importance in failures of an incremental collapse type. As a structure is deflected away from its original vertical equilibrium position, the increment in sidesway deflection under repetition of the same acceleration pulse will increase since the structure's available net yielding resistance against lateral inertial forces is considerably reduced by the P-Δ effect (Fig. 5). Accumulation of these increasing incremental deflections can lead to an instability phenomenon under a working load combination (gravity forces plus wind or minor earthquake). Figure 5 indicates that structural instability under working loads may be prevented or delayed by a reduction in the maximum tolerable story drift, by an increase in the yielding strength against lateral forces, or by a combination of these two possibilities. It should be noted, however, that the only advantage in increasing the initial stiffness without either modifying the yielding strength or maximum tolerable story drift will be a small increase in the energy absorption and energy dissipation capacity. Such an increase is illustrated in Fig. 7(a). This figure also indicates that an increase in initial stiffness without a reduction in tolerable story drift will lead to a considerable increase in ductility demands, and, therefore, greater structural damage. A reduction in the acceptable story displacement ductility will generally lower the danger of instability because such a reduction implies an increase in the required yielding strength of the structure which in turn usually requires a corresponding increase in the initial stiffness. The end result is a story drift at yielding equal to or less than that corresponding to a structure with a lower yielding strength, and a considerably smaller story drift at ultimate condition.

The behavior depicted in Fig. 5 suggests the approximate design method, illustrated in Fig. 7, for preventing or delaying the deformation instability under working load levels. The method is based on the assumption that maximum tolerable story drift, Δ_s^{MAX}, and story shear due to lateral working loads, S_s^W, are known. The total axial force acting on a story during severe seismic shaking is also assumed to be known since it depends only on the gravity forces acting above that story, P_s^G. Two different examples of possible inelastic behavior are considered in Fig. 7. If the mechanism deformation is of a perfectly plastic type, it will be sufficient to draw a line, BO', parallel to OA through point B [Fig. 7(a)]. If the mechanism deformation of the structural system is developed with some strain-hardening, it will be necessary first to locate point B'. Then drawing B'O' with a slope equal to the expected rate of strain hardening, intersection O' will give the mechanism yielding strength required, S_s^Y, as shown in Fig. 7(b). Comparison of Figs. 7(a) and 7(b) illustrates the advantage of having a structural system whose mechanism deforms with some strain hardening.

Experimental results [15] have shown that requirements for preventing instability of structural members depends on the desired level of ductililty. The larger the tolerable ductility, the more stringent the requirements should be. Under loading reversals, when the ductility value exceeds a certain limit, there is a sudden drop in resistance against instability, particularly in the case of reinforced concrete structures.

DAMAGEABILITY LIMIT STATES

LESSONS LEARNED FROM RECENT EARTHQUAKE DAMAGES. - Review of recent earthquake damage reveals that many buildings which did not collapse had to be either completely or partially demolished due to the high amount of nonstructural and structural damage which constituted failure. Numerous buildings whose structural systems did not undergo any significant structural damage, suffered such damage to nonstructural components as to render the entire building unfit for use. As previously pointed out, most present aseismic design methods focus on collapse (ultimate strength and displacement ductility) of the main structural system as the essential limit state. The main problem in applying such methods is in establishing the proper displacement ductility value. Selection of just one value cannot ensure that a structure will be safe and economical or that damage will remain within acceptable limits in all cases.

Although it is generally recognized that the most important single cause of damage is deformation, the types of deformations primarily responsible for damage to nonstructural components remain unclear. It has been argued that while lateral displacement ductility factors generally provide a good indication of structural damage, they do not adequately reflect damage to nonstructural elements [16]. Nonstructural damage is more dependent on the relative displacements (interstory drift) than on the overall lateral displacements. Aseismic design methods must incorporate drift (damage) control in addition to lateral displacement ductility as design constraints. Story drifts and drift ductility factors may also be useful in providing information on the distribution of structural damage, although conventionally computed story drifts are unreliable indicators of potential structural or nonstructural damage to multistory buildings. In some structures, a substantial portion of horizontal displacements results from axial deformations in columns. Story drifts due to these deformations are not usually a source of damage [Fig. 8(a)]. A better index of both structural and nonstructural damage, particularly for frames tightly infilled with partitions, is the tangential story drift index R. As schematically indicated in Fig. 8(b), this index is used to measure the shearing distortion within a story. For the displacement components shown in Fig. 8(c), the average tangential drift index is equal to $R = (u_3 - u_1)/H + (u_6 + u_8 - u_2 - u_4)/2L$.

Glogau [17] discussed the different types of deformations that could cause damage to nonstructural elements as well as formulated different damage control strategies. Broad damage mitigation strategies have also been discussed by Kost and associates [18].

DAMAGEABILITY. - Establishment of a proper failure criterion based on damageability requires development of a methodology for damageability as an inadmissible limit state under extreme (potentially catastrophic) environmental hazards of the whole building rather than that of the bare structure. Similar to other failure criteria for aseismic design, damageability limit states depend on the type of ground motions being generated. Not only should the intensity of these excitations be considered, but their general dynamic characteristics and their combinations with loads resulting from gravity forces and environmental effects should be accounted for. Damageability limit states can be considered as a category that bridges the gap between serviceability and safety against collapse. Although the primary causes of damageability with which we will be concerned are due to significant overexcitations (large deformational behavior of structural and nonstructural components), effects of service excitations on damageability should not be ruled out. Inadmissible limit states are usually described in terms of limiting the levels of structural response, e.g. maximum displacement, crack width, forces and moments. Although such structural responses may be related to the risk of life-loss, injury, and to economic losses resulting from damage, the relationship of structural response to damage and to socio-economic losses has not been clearly established. To facilitate the establishment of such a relationship, it is proposed to define indices of damageability for a given load or environment exposure history which can be used as an indicator of a limit state condition.

DAMAGEABILITY CRITERIA. - In considering damageability, three general types of damage must be distinguished: (1) local damage - limited to one or several typical elements; (2) global damage - overall damage in a particular event related to the total building; and (3) cumulative damage - overall damage resulting from a series of events, such as strong earthquakes followed by a series of aftershocks, or by other consequential or independent events such as fire, or some other combination of normal and catastrophic events.

Physical damage to both structural and nonstructural components is related to structural response characteristics. Recent advances in methods of structural analysis for complex nonlinear behavior under a variety of dynamic load conditions as well as under fire [19-21] and other environmental exposures provide a basis for investigating damageability. One problem encountered in these investigations involves the proper modeling of nonstructural components to study their interaction with structural models. Because there are no reliable data on the actual mechanical behavior of these components, it will be necessary to study the type and amount of deformation and/or forces that are required to produce different levels of damage in masonry, wood panels, gypsum boards, glazed openings, equipment, etc. Another difficulty in realistically assessing structural response and potential damage in existing structures subjected to earthquake is in properly evaluating the current state of the building at the time of the earthquake. Such evaluation involves considering the effects of (1) previous exposure to climatic environment (thermal changes or shrinkage), causing a state of residual stress or distress, and deterioration in structures due to aging and corrosion; (2) degradation in strength and stiffness caused by previous exposure to high winds, fires and/or earthquakes; (3) other disturbances or movements of the foundation; and (4) changes in strength and stiffness due to alterations, repair, or strengthening. Because any one of these conditions can significantly alter structural response, one of the problems that must be included in the study of damageability is the effect of variations in load and environmental histories, and the residual conditions in the structure (residual stress, cracking, corrosion, and other changes in stiffness or strength of the materials). Once the "present state" of a building has been properly assessed, and the mechanical (or mathematical) model is clearly described in terms of the intensity and characteristics of the ground motion, the response of a building (structural and nonstructural components) can be determined. A general evaluation framework, which is based on a sequence of basic procedures starting with the simplest models and employing more complex models as needed to achieve desired reliability, has been formulated [22]. This procedure is referred to as "screening."

Several procedures for evaluating earthquake safety of existing buildings were proposed following the 1971 San Fernando earthquake and have since been incorporated into practice [23]. These methods fall into two general categories. The first includes procedures which may be found in mandatory regulations, the second, proposals which focus on methodology and are published as technical reports or papers. These methods do not, however, address the problem of global or cumulative damage, nor do they provide a means for including nonstructural damage in an overall assessment of damageability.

DAMAGEABILITY INDICES. - An index of local damageability, D_i, for a given element i in a building exposed to a specified load or environmental exposure is defined here as the ratio of building response demand for this element (d_i) to its corresponding resistance capacity (c_i) that is, $D_i = d_i/c_i$, where capacity c_i is the limit value for building response without damage. Both structural and nonstructural elements should be considered in evaluating damageability index D_i. For the design of new buildings, values of d_i and c_i must consider randomness in loading demand as well as in "as built" condition determined by quality control during construction. With properly defined values of d_i and c_i, damage will occur when $D_i > 1$; when $D_i < 1$, no local damage should occur, and in this case, D_i should be assigned a value of zero.

Overall or global damageability index D_g may be defined as the sum of nonzero values of D_i, including structural and nonstructural components which might be damaged in a particular event of extreme exposure. Values of D_i must be weighted by an appropriate importance (life hazard, cost, etc.) factor, p_i, as $D_g = \Sigma_n p_i D_i$. The sum is taken over n damageable elements, including both structural and nonstructural components. Index D_g should be normalized to $\overline{D}_g$ in order to use the latter for comparing two buildings or two alternate designs of the same building. Several possible ways to accomplish this normalization should be explored. For example, $\overline{D}_g$ may be defined as $\overline{D}_g = D_g/\Sigma_n p_i$, or more appropriately as $\overline{D}_g = D_g/\Sigma_m p_i$, where n is the number of damageable elements, m is the total number of elements (both damageable and nondamageable), and $\Sigma_m p_i$ reflects some overall current value of a building.

The *cumulative damageability index*, D_c, may be defined as the sum of nonzero values of $p_i D_i$, including structural and nonstructural components which might be damaged as a result of a specified sequence of events, for example, fire exposure, repair of fire damage, strong earthquake, with specified strong aftershocks. Such factors can be taken into account in evaluating local damageability by introducing *service history influence coefficients* η_i (for demand) and χ_i (for capacity), which are also influenced by the randomness of these influences. Then $D'_i = \eta_i d_i/\chi_i c_i$, where D'_i is the current nonzero local damageability index which accounts for the assumed service history of a building. If N is the number of damageable components in such a case, then $D_c = \Sigma_N p_i D'_i$. Normalized value $\overline{D}_c$ can then be expressed as $\overline{D}_c = D_c/\Sigma_m p_i$. For old buildings, evaluation of the damageability index is further complicated by the significant influence that the service history of a building may have on the values of both demand and capacity (either increasing or decreasing these values), due to such factors as aging, change in use or occupancy or in socio-economic conditions (which would affect p_i values), structural and nonstructural modifications, fire damage and repair, corrosion, etc. The same problems exist for new buildings, due to the uncertainties associated with predicting future earthquakes. Then $D'_g = \Sigma_n p'_i D'_i$ and $D'_c = \Sigma_N p'_i D'_i$, where p'_i is the *current* importance factor (which may differ from the factor p_i used in the original design). Normalized values of $\overline{D}'_g$ and $\overline{D}'_c$ for existing buildings can be defined similarly to $\overline{D}_g$ and $\overline{D}_c$ values for new buildings. The larger the value of $\overline{D}$ or $\overline{D}'$, the greater the overall damageability index of a building. When $\overline{D}$ or $\overline{D}'$ exceeds some specified limit value, the damageability risk is too great and the building should either be redesigned or strengthened, or demolished.

DAMAGEABILITY AS FAILURE CRITERION. - The general philosophy of developing a method and criteria for assessing damageability has been presented, but the methodology for evaluating the different damageability indices are still undergoing development [23]. One of the main problems encountered in developing such methodology is in defining reliable procedures for calculating the values of d_i, c_i, p_i, η_i, χ_i, and p_i'. Quantification of damageability limit states will require extensive investigation of the mechanical behavior of nonstructural elements, or, what Kost et. al. [18] have termed, EFS (enclosure, finish, and service systems) components. With the findings from such studies, it will be possible to develop a conceptual model for analyzing the dynamic behavior of entire soil-structure systems. Implementation of the model in damageability limit state studies will enable guidelines for assessing failure criteria in aseismic design to be formulated.

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TABLE 1 LATERAL INTERSTORY DRIFT INDEX LIMITATIONS FOR ESSENTIAL FACILITIES

U.S. VETERAN ADMINISTRATION HOSPITALS(1)	1976 UBC(2)	NEW ATC-3 PROPOSAL(1)	MEXICO FEDERAL DISTRICT(1)	NEW ZEALAND(1)
0.0078	0.005	0.01	0.05	0.006(c)
0.0026(a)	0.01(b)			0.01(d)

(1) Maximum value considering inelastic deformations.

(2) Maximum value based on code prescribed forces at service level.

- (a) For glazed openings.
- (b) Equipment must remain in place and be functional.
- (c) When nonstructural components are not separated from the structure.
- (d) When nonstructural components are separated from the structure.

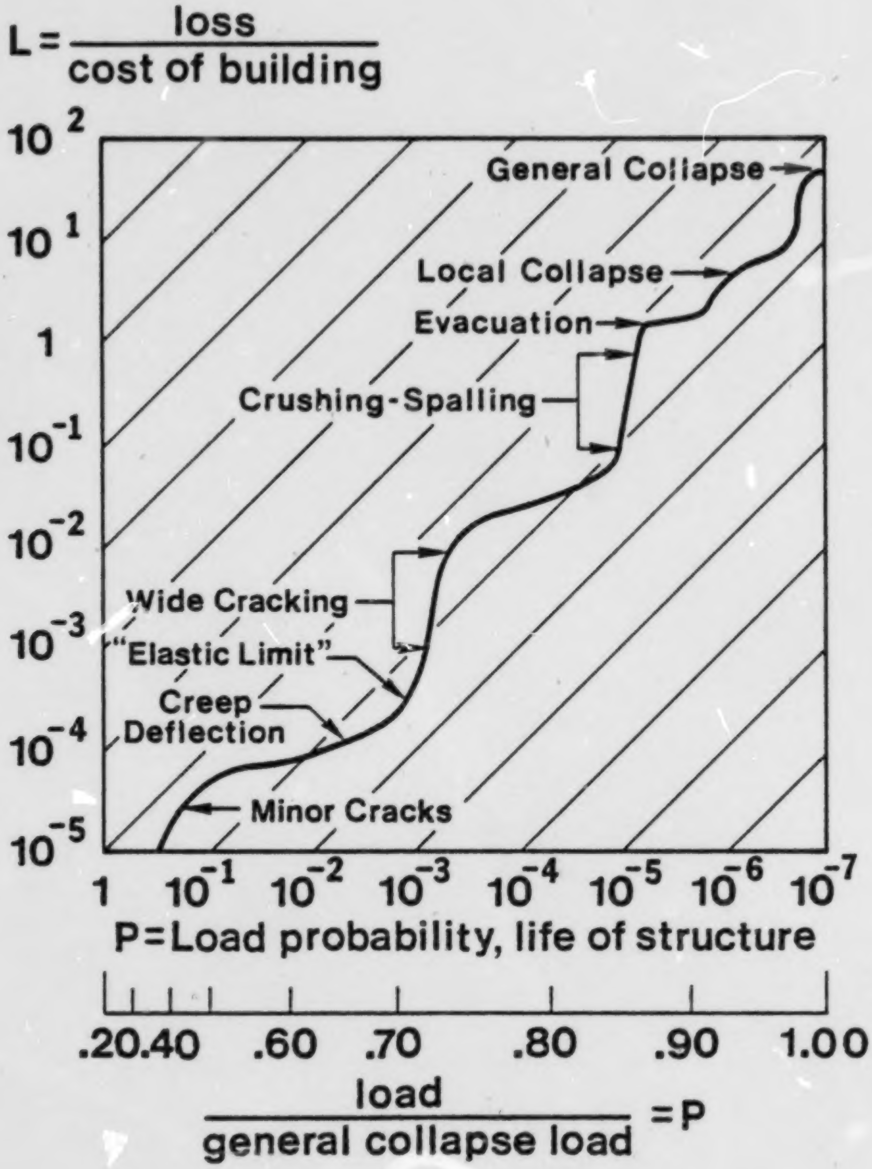


FIGURE 1 ASSESSMENT OF MEAN LOSSES VS. LOAD PROBABILITIES DURING THE LIFE OF STRUCTURE [5]

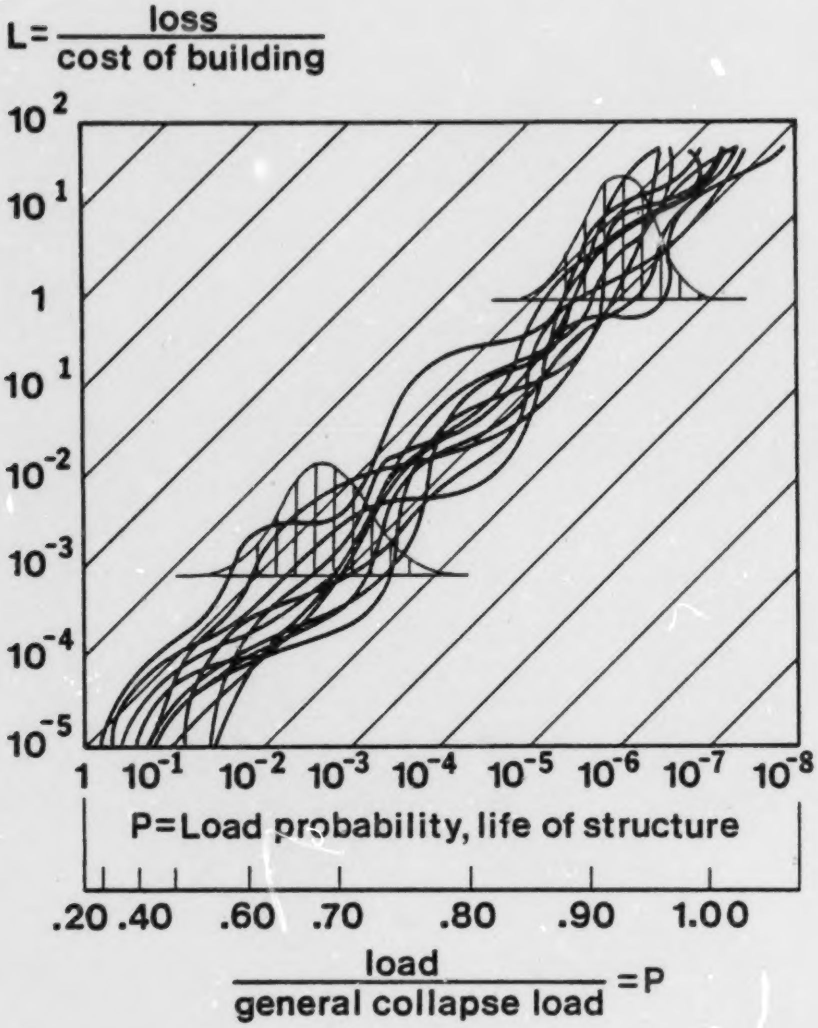


FIGURE 2 DISTRIBUTION OF LOSSES VS. LOAD PROBABILITIES DURING THE LIFE OF STRUCTURE [6]

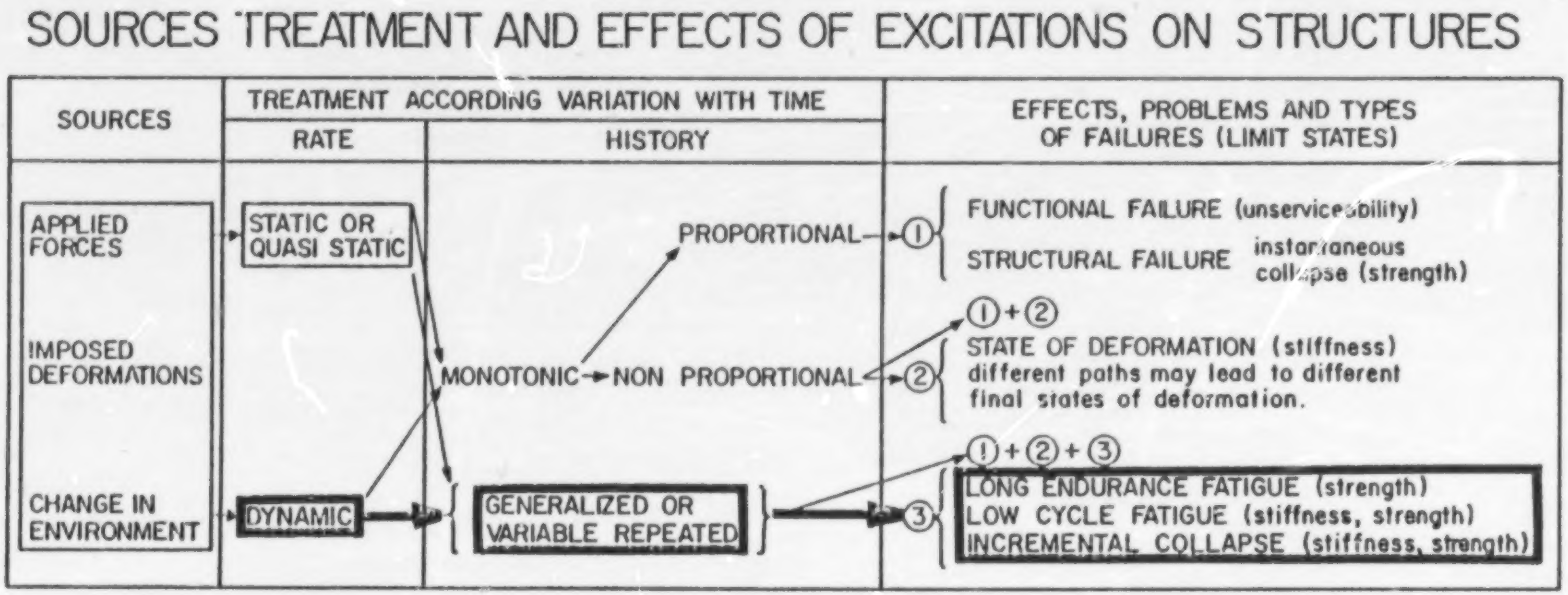

SOURCES TREATMENT AND EFFECTS OF EXCITATIONS ON STRUCTURES
SOURCES
TREATMENT ACCORDING VARIATION WITH TIME
RATE
HISTORY
EFFECTS, PROBLEMS AND TYPES OF FAILURES (LIMIT STATES)
APPLIED FORCES
IMPOSED DEFORMATIONS
CHANGE IN ENVIRONMENT
STATIC OR QUASI STATIC
DYNAMIC
MONOTONIC
PROPORTIONAL
NON PROPORTIONAL
GENERALIZED OR VARIABLE REPEATED
①
FUNCTIONAL FAILURE (unserviceability)
STRUCTURAL FAILURE instantaneous collapse (strength)
① + ②
②
STATE OF DEFORMATION (stiffness) different paths may lead to different final states of deformation.
① + ② + ③
③
LONG ENDURANCE FATIGUE (strength)
LOW CYCLE FATIGUE (stiffness, strength)
INCREMENTAL COLLAPSE (stiffness, strength)

FIGURE 3 SOURCES, TREATMENT, AND EFFECTS OF EXCITATIONS ON STRUCTURES

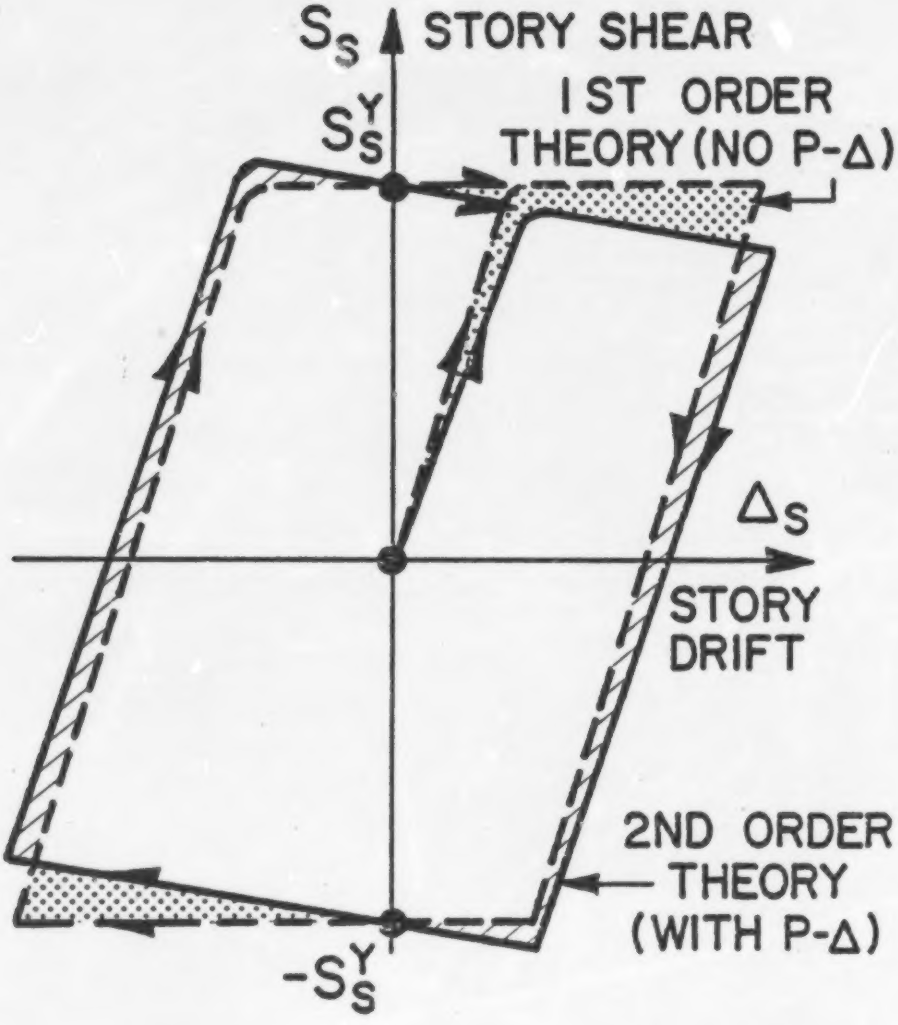

S_S STORY SHEAR
S_S^Y
1ST ORDER THEORY (NO P-Δ)
Δ_S
STORY DRIFT
2ND ORDER THEORY (WITH P-Δ)
$-S_S^Y$

FIGURE 4 EFFECT OF P-Δ ON LOW-CYCLE FATIGUE

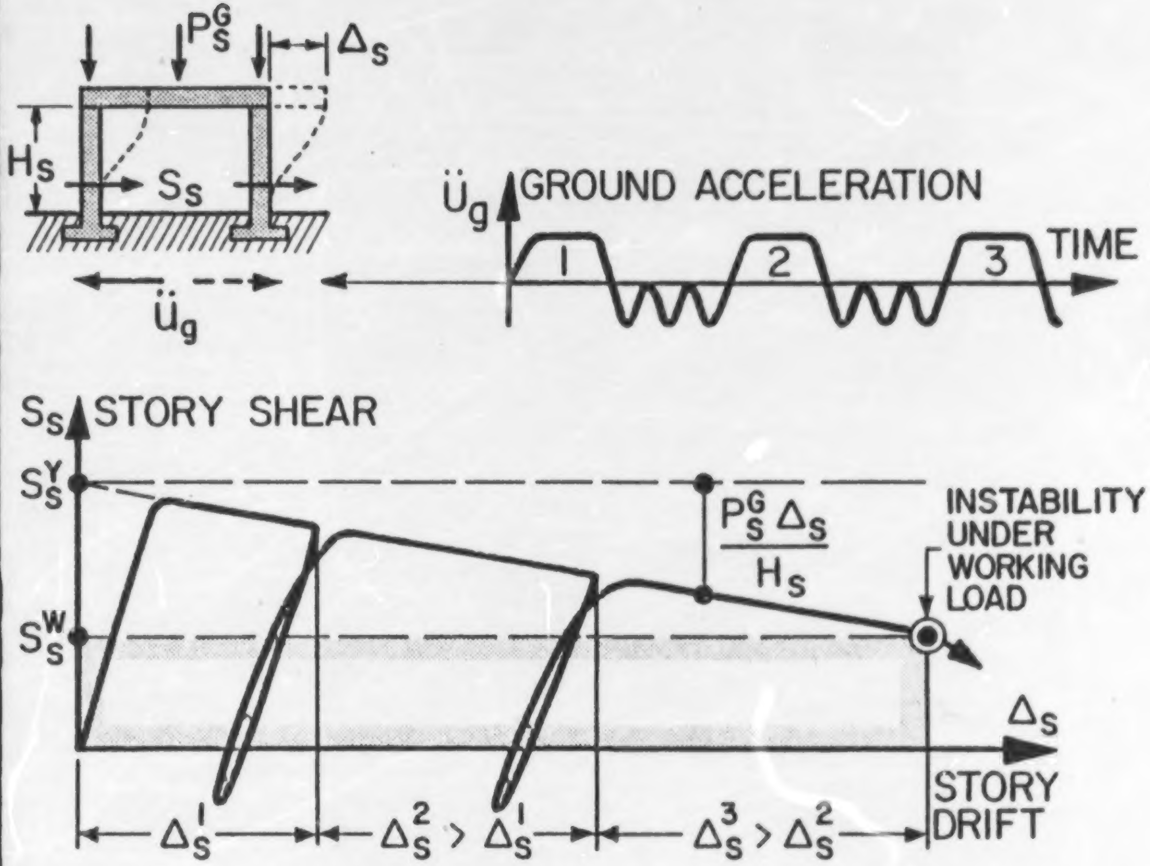

P_S^G
Δ_S
H_S
S_S
$\ddot{u}_g$
$\ddot{U}_g$ GROUND ACCELERATION
TIME
1
2
3
S_S STORY SHEAR
S_S^Y
S_S^W
$\frac{P_S^G \Delta_S}{H_S}$
INSTABILITY UNDER WORKING LOAD
Δ_S
STORY DRIFT
Δ_S^1
$\Delta_S^2 > \Delta_S^1$
$\Delta_S^3 > \Delta_S^2$

FIGURE 5 INCREMENTAL COLLAPSE

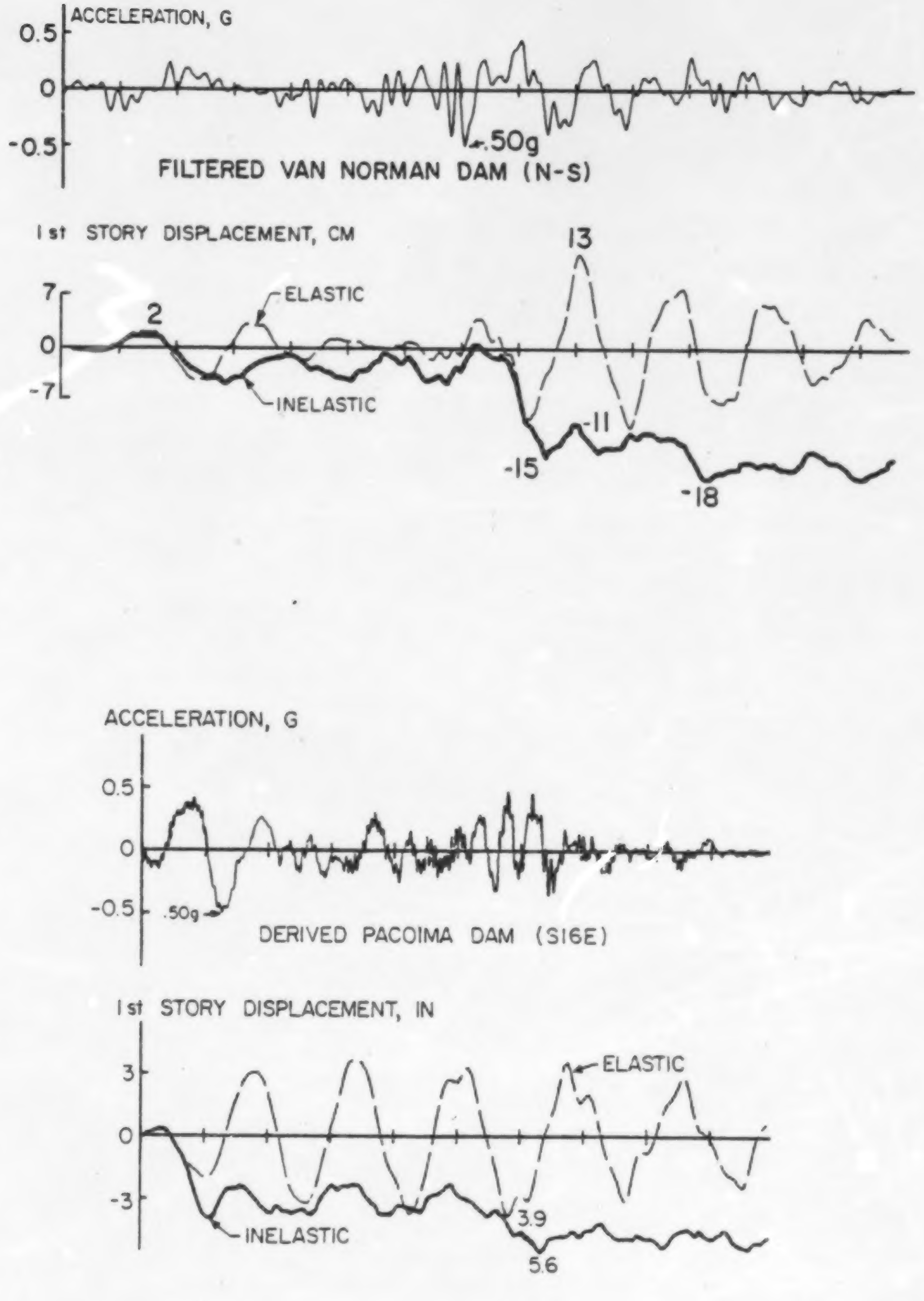
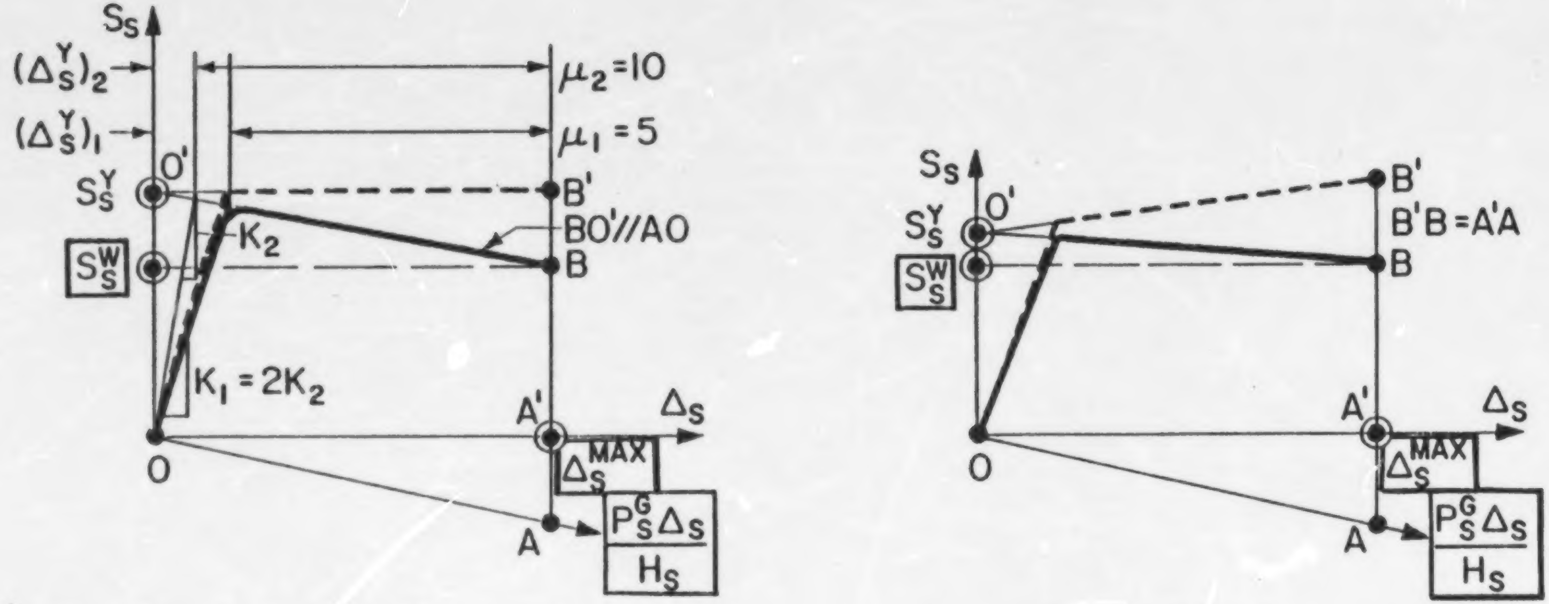


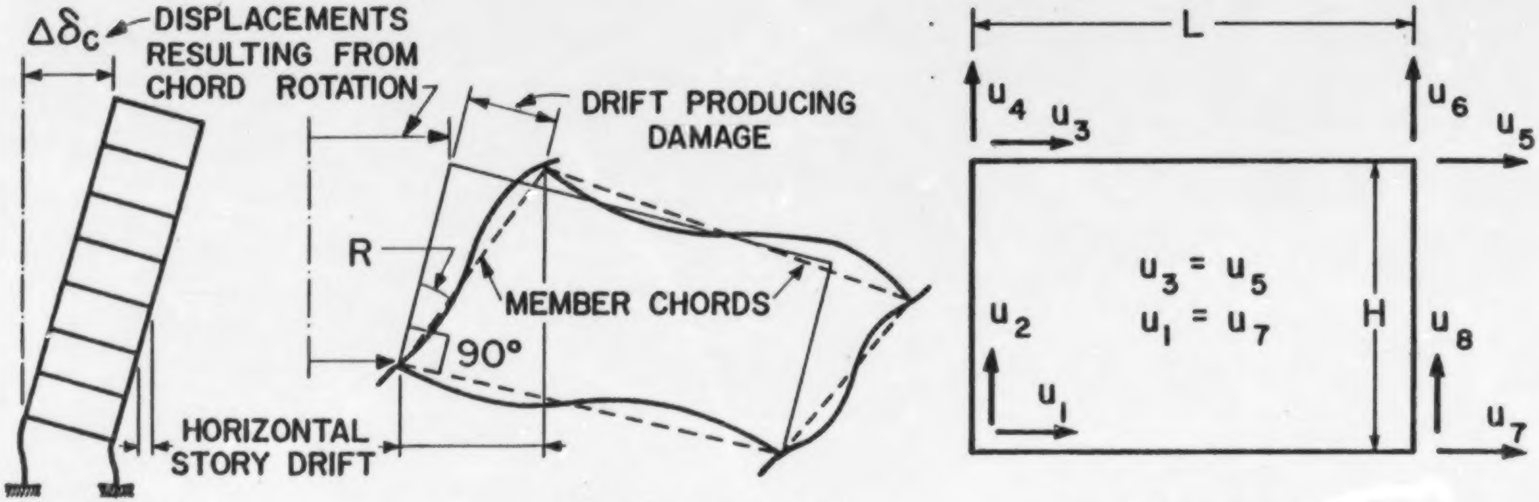
FIGURE 6 FIRST-STORY DISPLACEMENT TIME-HISTORY RESPONSE [14]



(a) ELASTIC-PERFECTLY PLASTIC BEHAVIOR

(b) STRAIN-HARDENING BEHAVIOR

FIGURE 7 DETERMINATION OF REQUIRED YIELDING STRENGTH TO AVOID INSTABILITY DUE TO P-Δ


ΔδC
DISPLACEMENTS RESULTING FROM CHORD ROTATION
HORIZONTAL STORY DRIFT
DRIFT PRODUCING DAMAGE
R
MEMBER CHORDS
90°
L
u4
u3
u6
u5
u3 = u5
u1 = u7
H
u2
u8
u1
u7

(a) DRIFTS DUE TO AXIAL FORCES IN FIRST STORY COLUMNS

(b) DRIFT DUE TO STORY DEFORMATION

(c) DISPLACEMENT COMPONENTS FOR COMPUTING R

FIGURE 8 STORY DRIFT

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